U.S. Army Corps of Engineers
Charleston District

APPENDIX B

CHARLESTON HARBOR POST 45
CHARLESTON, SOUTH CAROLINA

Geotechnical

03 October 2014

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I. INTRODUCTION

1.1 Purpose.

Appendix B provides documentation of the geologic conditions that influence the feasibility of the proposed harbor deepening. Geotechnical facts, assumptions, and interpretations used by the PDT are presented in this appendix. Interpretations are based upon established geologic conditions, new and existing borings, washprobes and geophysical surveys.

1.2 Organization.

The regional geologic setting and stratigraphic framework are addressed in Chapter II. Hydrogeology and dredging impacts to groundwater resources are addressed in Chapter III. The bulk of Appendix B focuses on the subsurface conditions within the upper and lower harbor and the entrance channel. Chapter IV describes the materials present within the upper and lower harbor sections based upon interpretation of historical boring logs. Chapter V presents the results from a subsurface investigation conducted within the entrance channel from November 2012 to September 2013. This chapter describes the attempts to delineate the location, extent, and strength of bedrock within the entrance channel, and provides an assessment of its dredgeability. Chapter VI presents the results from a preliminary seepage and stability analysis for Clouter Creek Disposal Area.

The following Attachments to Appendix B have been removed from the hardcopy document, but are available to download in PDF format from the Charleston District:

- Attachment B-1 Boring Logs Upper and Lower Harbor[1]
- Attachment B-2 Entrance Channel Boring Logs[2]
- Attachment B-3 Entrance Channel Soils Gradation Data[3]
- Attachment B-4 Entrance Channel Rock Strength Data[4]
- Attachment B-5 Entrance Channel Top of Rock Surface Data[5]

[1] http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/B-1%20Boring%20Logs%20Upper%20and%20Lower%20Harbor.pdf
[2] http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/B-2%20Entrance%20Channel%20Boring%20Logs.pdf
[3] http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/B-3%20Entrance%20Channel%20Soils%20Gradation%20Data.pdf
[4] http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/B-4%20Entrance%20Channel%20Rock%20Strength%20Data.pdf
[5] http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/B-5%20Entrance%20Channel%20Top%20of%20Rock%20Surface%20Data.pdf

II. REGIONAL GEOLOGY

2.1 Geologic Setting

The Charleston Harbor project site lies within the South Carolina Coastal Plain, which forms an embayment south of the Cape Fear Arch (Figure B-1). Deep crustal faulting associated with Late Triassic rifting produced a subsiding depositional basin, which contains Cretaceous and Tertiary sediments (Harris et al., 1979; Horton and Zullo, 1991; Harris et al., 2005). The stratigraphy of the South Carolina Coastal Plain consists of partially consolidated, unconformity bound, southeast dipping estuarine-marine shelf Tertiary deposits, which are overlain by unconsolidated Quaternary barrier and nearshore deposits. Superimposed upon this stratigraphy are escarpments and terraces that were carved into the strata as a result of interglacial sea-level fluctuation that began as early as 240,000 years ago (Weems and Lemon, 1993). The development of the modern shoreface with its barrier islands, inlets, and intertidal waters, was strongly influenced by the geology and topography of resistant strata (Harris et al., 2005).

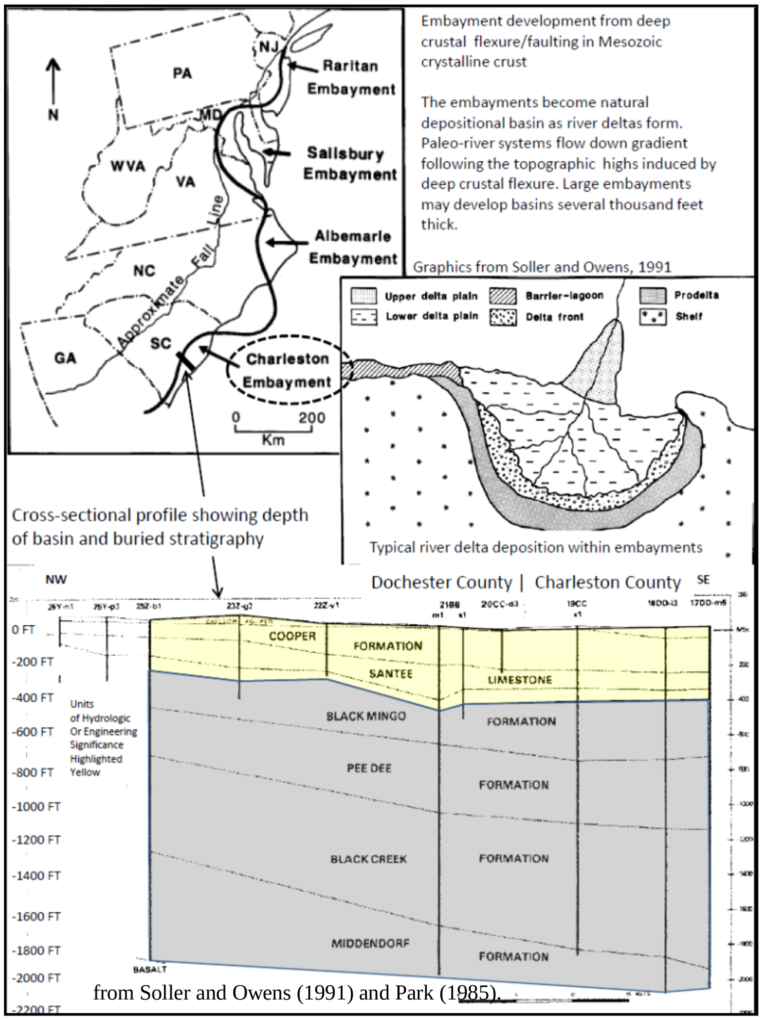

Figure B-1. Regional geologic setting of the Charleston Embayment.

2.2 Stratigraphy

The stratigraphic units that are most significant to the project are Tertiary in age. Specifically, these units are the Black Mingo Group, Santee Limestone, Cooper Formation, Edisto Formation, and Marks Head Formation. These stratigraphic units are relevant because of their hydrogeologic properties, or their occurrence within the project site (Figure B-2). The units are lithologically distinct from each other and are disconformity bound. Pre-Cretaceous basement crystalline rocks and Cretaceous-age strata belonging to the Middendorf, Black Creek, and Pee Dee Formations lie at elevations of -3000 to -200 feet mean sea level (msl), and are too deeply buried to be of engineering concern for this project. Quaternary units are generally found as surficial unconsolidated deposits along the shoreline and inland areas.

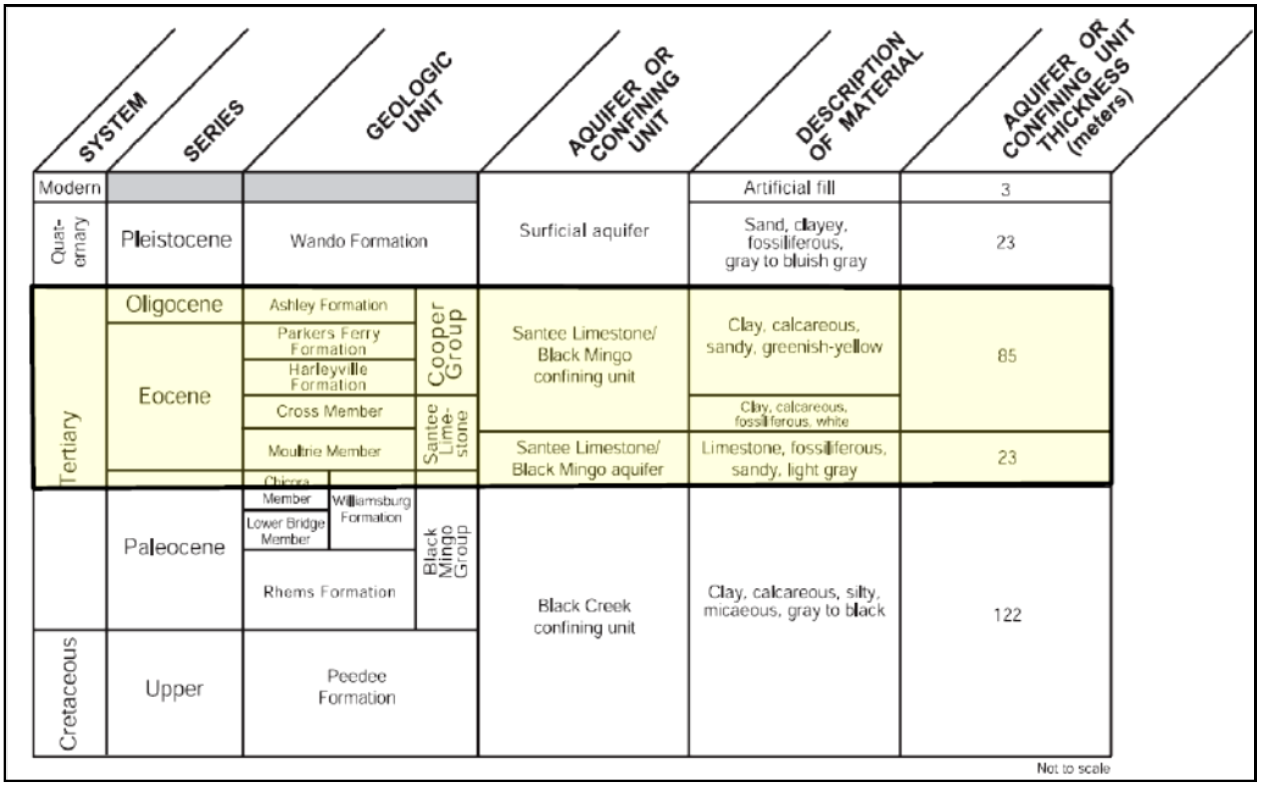

Figure B-2. Project relevant stratigraphic & hydrogeologic units, from Petkewich et al. (2004)

2.2.1. Black Mingo Group

The Black Mingo Group was named for exposures of mudstone along the Black River and Black Mingo Creek by Sloan (1907). Other agency and private drill core data indicates that the unit is heterogeneous and comprised of interbedded sequences laminated clay, mudstone, sand and limestone. The base of the unit is predominantly composed of mudstone and silty-clay interbedded with calcareous sands with occasional limestone, where as the top of the unit is predominantly fossiliferous limestone interbedded with quartz sand and occasional clay (Bybell et. al., 1998; Edwards et al., 1999). The Black Mingo sediments are generally a mixture of clastic detrital material and volcanic ash that were deposited within inner shelf and marginal marine environments during the Late Paleocene to Early Eocene. Outcroppings of the formation occur in Monck’s Corner and surrounding counties, and it dips south-southwest into the subsurface to a depth of -600 ft. msl below southern Charleston County (Park, 1985).

2.2.2. Santee Limestone Formation

The Santee Limestone is named for exposures that occur along the Santee River in South Carolina, where it underlies the Cooper Group (Sloan, 1908). The Santee Limestone is creamy-white to gray, fossiliferous, glauconitic and has sand to mud-supported matrix. The unit is middle to late Eocene in age and disconformity bound (Park, 1985). Two members are generally recognized within the Santee Limestone; the middle Eocene Moultrie Member and middle to late Eocene Cross Member (Figure B-3). The Moultrie Member of the Santee Limestone is approximately 7-feet thick from recovered drill cores and the limestone matrix tends to be coarse-grained, bioturbated, moldic and sandy. The Cross Member is significantly thicker (39-feet thick from drill core) with a finer-grained, clayey matrix. Deposition of the Santee Limestone occurred 45-41 million years before present, when shallow open marine-shelf environments were drowned and transformed into deeper outer continental shelf environments (Petkewich et. al., 2004). The Santee Limestone is exposed in surficial exposures located along a 5-mile wide belt that extends across northern Dorchester, Berkeley, and Charleston Counties, and it dips into the subsurface towards the south-southeast (Figure B-3). The top of the formation is lies at -300 feet msl beneath Charleston Harbor. The unit thickens southwestward from 20-feet thick near Lake Moultrie to over 260-feet thick beneath Edisto Island (Park, 1985).

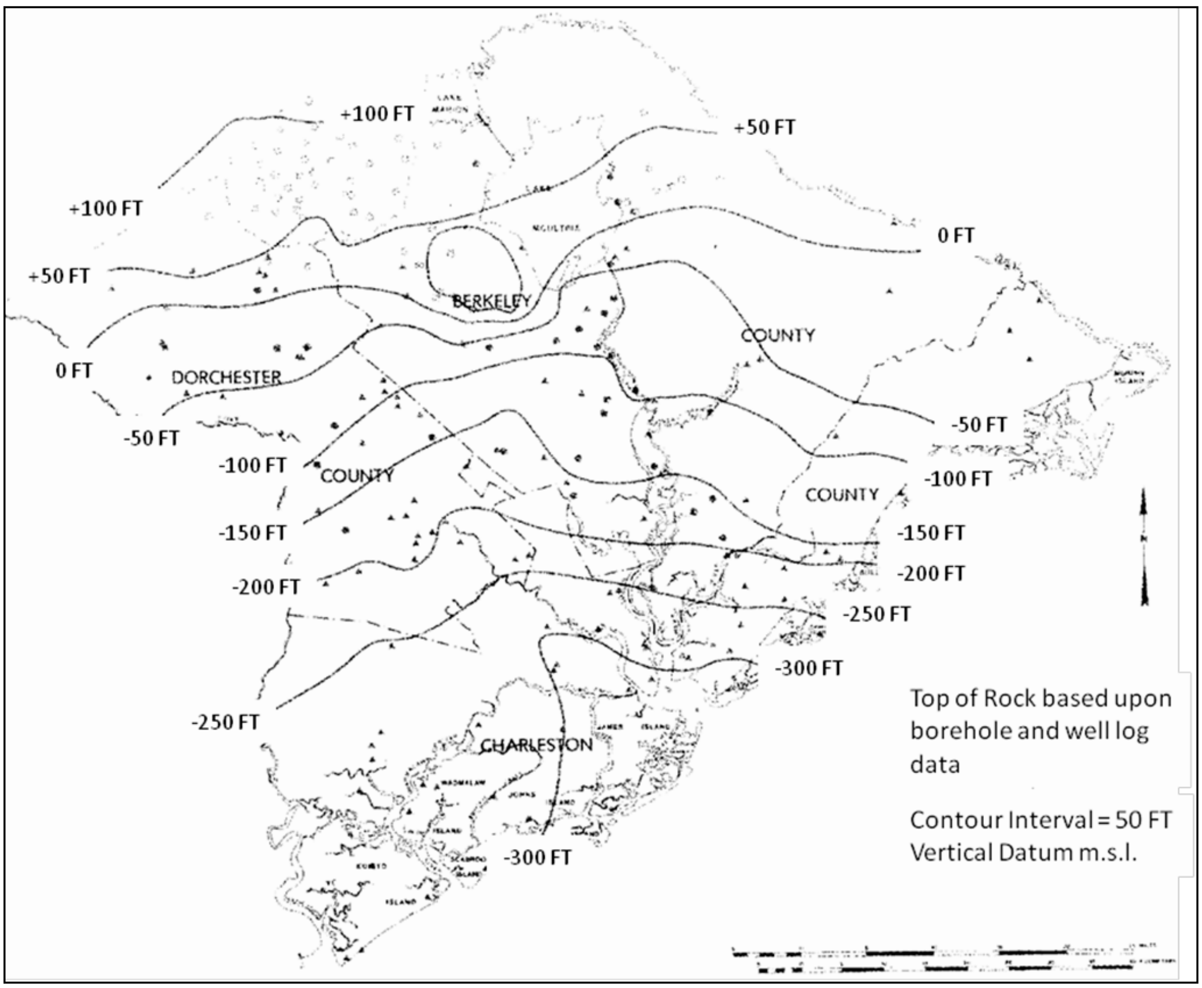

Figure B-3. Structural contour map showing top of Santee Limestone, from Park (1985).

2.2.3. Cooper Formation

The Cooper Formation was originally termed "Cooper Marl" by Toumey (1848) for exposures of soft, very fine grained, impure carbonate material found along the Cooper River and Ashley Rivers in South Carolina. This unit has been described by various workers in surficial exposures within the coastal plains of North Carolina, South Carolina and Georgia (Toumey, 1848; Cooke, 1952; Malde, 1959; Weems and Lemmon, 1993; Weems and Lewis, 2002)). Carbonate-rich sections of the unit were extensively studied and served as a source for agricultural lime production between 1867 and 1920. Upland exposures of the Cooper Formation are described as consisting of fine-grained calcareous foraminiferal shell material (Malde, 1959; Gohn et. al., 1977; Park, 1985). In contrast, soil borings, grab samples, and surficial exposures of the Cooper Formation within Charleston Harbor, resemble a consolidated and impermeable soil that ranges in composition from stiff clayey silt to dense silty sand. Weems and Lemon (1993) indicated that the Cooper Formation (Toumey, 1848) actually consists of a composite sequence of variably consolidated silt and clay, soft clayey and sandy limestones, and phosphatic deposits of Eocene-Oligocene age (Park, 1985; Weems and Lemon, 1993).

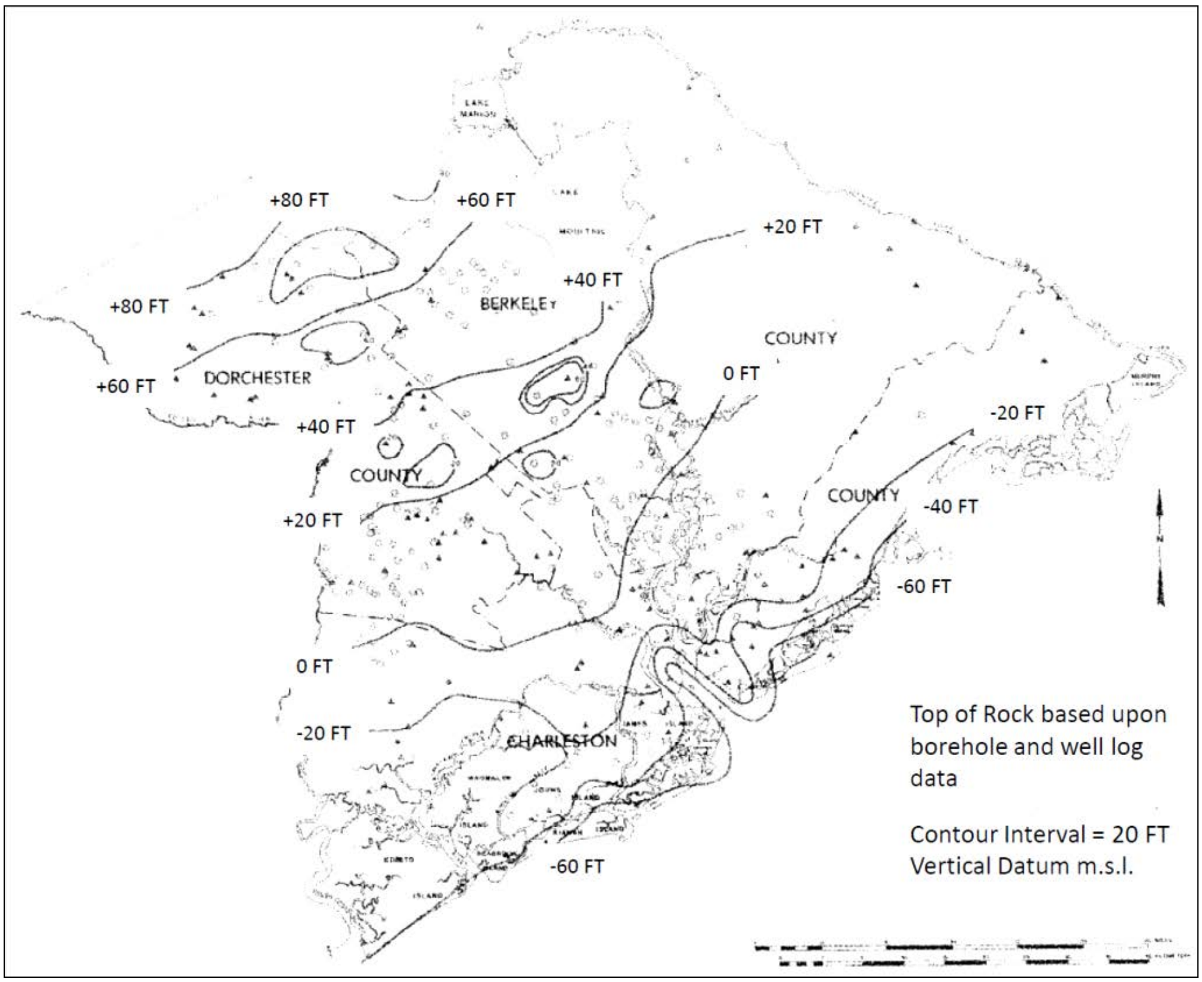

Figure B-4. Structure contour map showing top of Cooper Formation, from Park (1985).

However, the term "Cooper Formation" (Toumey, 1848) is the most recognized name for the unit by the PDT, and is hereby informally extended to encompass the Ashley and Chandler Bridge Formations described by Weems and Lemon (1993) and Weems and Lewis (2002). Therefore, for the purposes of this study, the Cooper Formation will be used to describe the stiff to very stiff, dense, impermeable fine-grained strata that forms the foundation of much of the harbor bottom.

Structural contour maps indicate that the Cooper Formation dips into the subsurface toward the south-southeast at a gradient of 8ft/mile (Figure B-4). Beneath the city of Charleston, the top of the Cooper Group lies at an elevation of -20 feet m.s.l, but due to the dipping gradient and high subsurface relief, it plunges to a depth of -60 feet msl near mouth of the harbor. Parks (1985) determined that the stratum thickens to 280 feet beneath Charleston Harbor (Figure B-5).

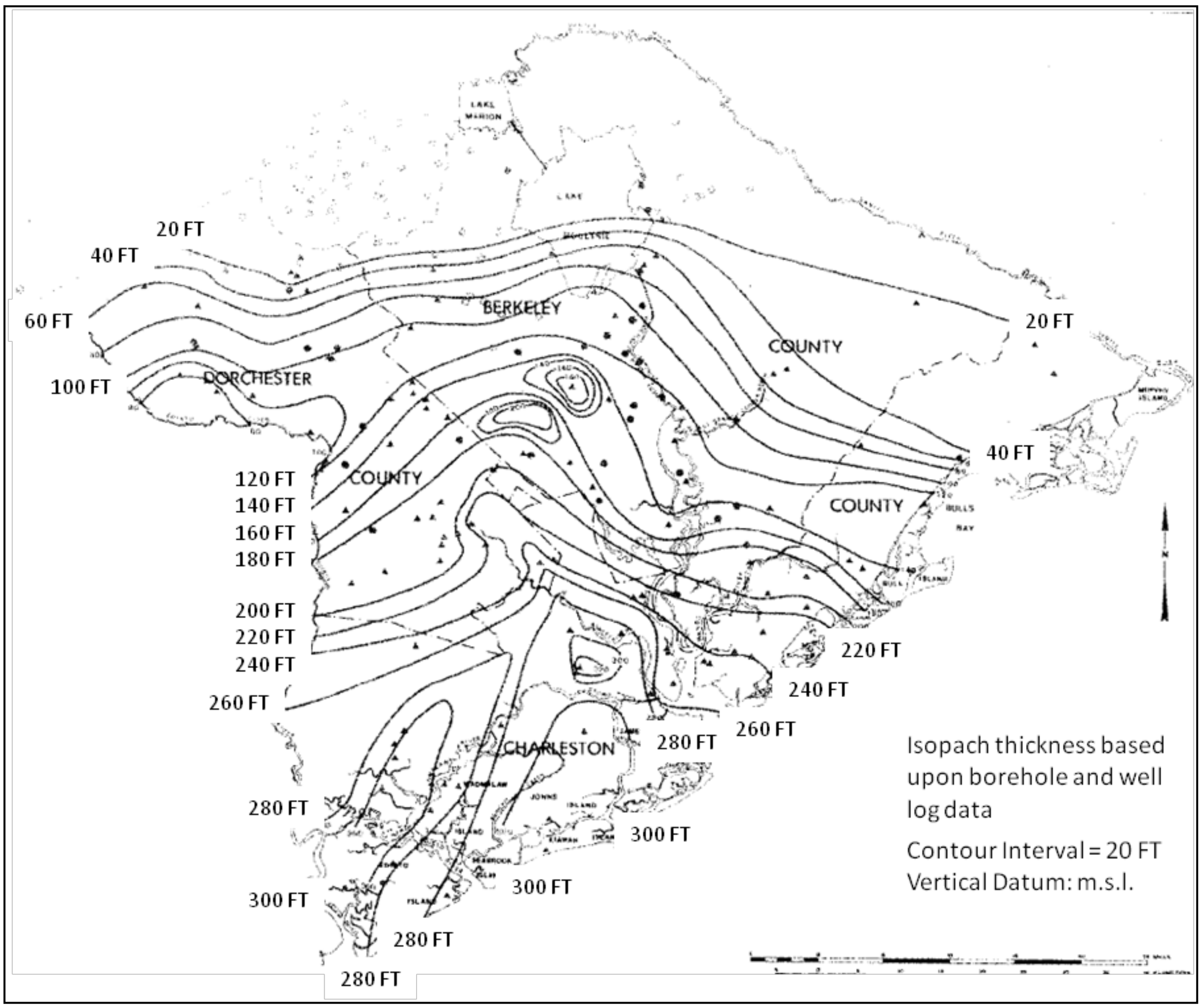

Figure B-5. Isopach map showing thickness of the Cooper Formation, from Park (1985).

SCDNR describes the unit as a stiff, partially consolidated, calcareous, silty-clay (SCDNR, Doars, personal communication, 2012). USACE drilling logs that penetrate into the Cooper Group describe the soil as a stiff to very stiff or hard, brown to greenish colored, clayey inorganic silt to silty clay, which had been classified as (MH, CH, ML, MH-CH, and ML-CL) per ASTM D2487. This material appears to grade into and out of medium dense clayey sand and stiff to hard lean clay. Brainard et al. (2009) state that historically, tunnel construction in Charleston area was conducted within the Cooper Formation (Cooper Marl) because of the unit's optimal engineering characteristics of low permeability, stiffness, and the relative ease by which it can be excavated. However, several water-bearing sand lenses 30-feet thick have been encountered during tunnel excavation (Brainard et al., 2009).

The Cooper Formation is comprised of at least four major subunits; the Eocene Harleyville and Parkers Ferry Formations, and the upper Oligocene Ashley and Chandler Bridge Formations. Collectively, these units were deposited in shallow to open marine environment 30 to 38 million years ago. The strata range in composition from phosphatic clay, to sandy limestone, to fine-grained silty-clayey phosphatic sand (Ward et. al., 1979; Weems and Lemon, 1984; Weems and Lemon, 1993). Harris et al. (2005) verified the top of the Cooper Formation at elevation -60 feet msl by seismic profile in the vicinity of Folly Island (Figure B-6).

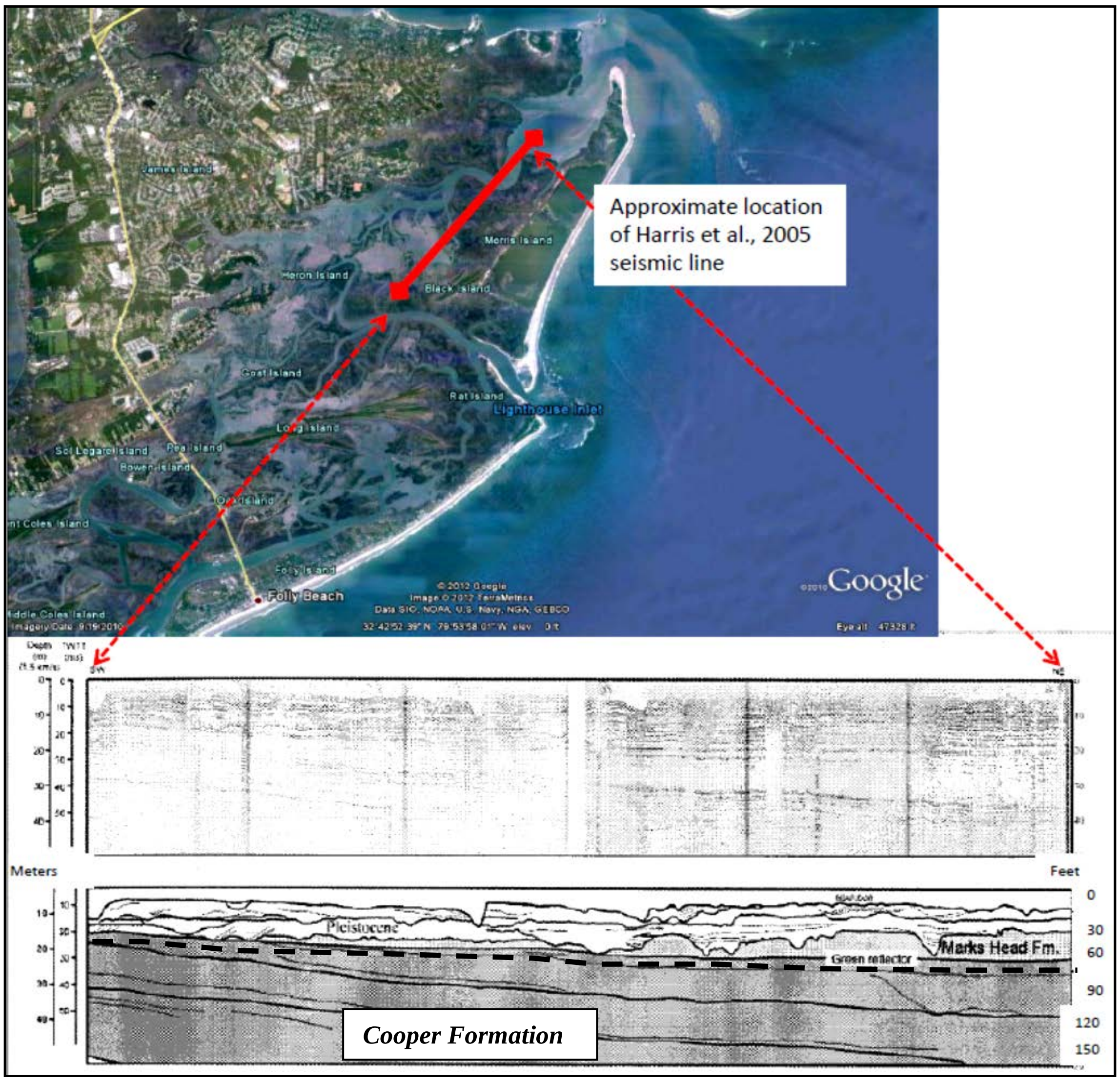

Figure B-6. Seismic profile south of Charleston Harbor, from Harris et al. (2005)

2.2.4. Edisto Formation

Ward et al., (1979) applied the name "Edisto Formation" to sandy-shelly limestones of early Miocene age that unconformably overlie the Cooper Formation in southern South Carolina. Weems and Lemon (1993) describe the unit as consisting of light gray, fine-grained calcareous sand to quartzose calcarenite[6], with locally abundant pelecypod shells. The Edisto Formation is generally composed detrital weakly cemented sand, gravel, and shell hash. The unit was deposited in a shallow marine environment 24 million years ago during the Miocene-Oligocene time. Weems and Lemon (1993) report the occurrence of phosphate nodules in land borings but none occur in offshore borings. The Edisto Formation unconformably overlies the Cooper Formation within the study area, however the stratigraphic contact was not observed in drill core. The thickness of the unit is unknown.

2.2.4. Marks Head Formation

The Marks Head Formation is described as fine-grained, quartz-phosphate sand, Miocene in age. The unit is known to lie unconformably atop the Cooper Formation and was deposited in shallow-brackish water conditions. Weems and Lemon (1993) indicate that the unit is discontinuous and only occurs in the near subsurface northeast of Charleston, beneath Mount Pleasant and Sullivan Island. South of Charleston, the unit is present from -30 to -60 feet msl and is no more than 30-feet thick (Harris et al., 2005) . Marks Head Formation dips into the subsurface south and east from surficial outcroppings north of Charleston (Weems and Lewis, 2002). The base of the unit is present at elevations -20 to -80 feet msl near Charleston Harbor. The shallowest occurrence of this stratum is likely to occur within the Ashley River near Duck Island and north of the confluence of the Cooper and Wando Rivers.

2.2.4. Quaternary Units

Nearly all of the surficial deposits in the Charleston area are Quaternary in age, and they unconformably overlie the Tertiary strata. These sediments were deposited during sea-level fluctuations caused by multiple interglacial cycles throughout the Pleistocene. At least five different sea-level stands are recognized near Charleston, based upon the presence of Pleistocene-aged terrace deposits and erosional shoreline escarpments. These geomorphologic features lie as far as 45-miles inland and mimic the shape of the modern coastline (Weems and Lemon, 1993; Harris et al., 2005). The Quaternary age strata generally consists of interbedded sequences of clay, clayey to clean quartz sand, and fossiliferous sand that may be capped by peat, clean sand, or tidal marsh deposits (Weems and Lemon, 1993).

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[6] Calcarenite is a type of limestone that is composed predominantly (> 50 percent) of detrital (transported) sand-size (0.0625 to 2 mm in diameter), carbonate grains. This material is derived from corals, shells, fragments of older limestones, and other carbonate clasts. Calcarenite is the carbonate equivalent of a sandstone. They can consist of grains of carbonate that have accumulated either as coastal sand dunes (eolianites), beaches, offshore bars and shoals, turbidites, or other depositional settings. Reference: http://en.wikipedia.org/wiki/Calcarenite

III. HYDROGEOLOGY & DREDGING IMPACT ASSESSMENT

3.1 General

The chapter presents an inventory of the groundwater resources that are present within Charleston, South Carolina, and their susceptibility to impact from dredging activities associated with proposed Post 45 Charleston Harbor Deepening Project. This project will deepen the current harbor in order to handle a new class of container vessels that carry a 50-foot draft. The proposed project will further deepen the entrance channel from 52 feet to 58 feet and the harbor interior from 45 to 56 feet, referenced to Mean Lower Low Water (MLLW). In order to predict the effects the new dredging will have on freshwater resources of the Charleston area, it is essential to identify where most of the population receives its potable water, the primary aquifers that are at risk, and potential impacts to drinking water supply.

3.1.1. Purpose

The primary hydrologic concern for any mass excavation or dredging is the unforeseen excavation into a confined aquifer system that will result in loss of hydraulic head, and loss of groundwater supply.

The purpose of this chapter is to provide an inventory and document the groundwater resources that are present within the Charleston Area and demonstrate their sensitivity to dredging impact, and if impacted, what the potential effects are to the public.

It is shown in this chapter that little to no impact to the water supply of Charleston and surrounding areas by deepening of the existing ship channel. This is done by presenting relevant stratigraphic/hydrogeologic data, water resource information, and well data, and comparing it to a maximum dredge depth. Open-source data indicates that the City of Charleston receives much of its drinking water from reservoir and surficial rivers and that the major producing aquifers are deeper than the maximum dredge depth.

3.1.2. Data Collection Efforts

Data collection was limited to published data including groundwater reports, geologic maps and well borings. These data were compared to a buffer zone that extends to -60 feet MLLW maximum elevation, which is considered a conservative depth for this evaluation. No new drilling or exploratory work was conducted to assess groundwater conditions; as such, this report reflects the general subsurface conditions as they are presently understood through available documentation.

3.1.3. Groundwater Modeling

No modeling was conducted for this assessment. Well boring data were plotted and queried in ArcGIS in an attempt to illustrate data trends.

3.2 Hydrogeologic Units

The stratigraphic units that comprise the South Carolina Coastal Plain are divided into a series of aquifers and confining units based upon their respective water-bearing characteristics. The six major aquifers beneath Charleston, SC are shown in Figures B-2 and B-3. These are from oldest to youngest; the Cretaceous Cape Fear aquifer, the Late Cretaceous Middendorf and Black Creek aquifers, the Paleocene-Early Eocene Black Mingo (sand aquifer), the Mid-Late Eocene Floridian (Santee-Cooper) aquifer, and the Quaternary surficial aquifer (Petkewich et. al., 2004; Aucott and Speiran, 1985). The Late Cretaceous Peedee aquifer lies unconformably atop the Middendorf and Black Creek aquifers; however, water quality and production from this aquifer is poor according to Parks (1985). Porous limestone and/or sandy strata that are capable of storing and transmitting groundwater to wells and springs comprise most of the aquifers, with exception to the water-producing strata of the Black Mingo. All of the deep aquifers are confined by fine-grained limestone or clayey strata. The Quaternary surficial aquifer is unconfined. Figure B-2 is provided in order to illustrate the general correlation between the aforementioned stratigraphic units and the major aquifers present beneath the Charleston area.

3.2.1. Cretaceous Aquifers

The Cape Fear, Middendorf, and Black Creek aquifers are the most voluminous water-bearing aquifers beneath South Carolina Coastal Plain, and are part of the larger Southeastern Coastal Plain aquifer system. These aquifers are comprised of Late Cretaceous terrigenous clastic sediments that were deposited in large river deltaic environments (Park, 1985; Miller, 1990). These aquifer systems are very deep; well screens set to this aquifer system are typically set between -800 to -2,800 feet m.sl. The groundwater flows under artesian conditions and has yields that range from 250 to 2000 gallons per minute (g.p.m.). The water is highly mineralized with variable concentrations of sodium bicarbonate, chloride, sodium and fluoride. Salinity increases with proximity to the coast. Given its relative depth and high mineral content, this aquifer system is not used for domestic (household) consumption within Charleston County. This aquifer system is generally accessed by the surrounding inland counties for irrigation, industrial, and public sector use.

3.2.2. Paleocene-Early Eocene Aquifer and Aquiclude

The lower 150-250 feet of the Black Mingo Group is impermeable and consists of interbedded silty clay and clayey sand. This forms an effective confining unit between the Cretaceous aquifer and Tertiary Floridian aquifer systems (Park, 1985; Park, online report: NOAA-NERRS ACE Basin Characterization). The upper 100 feet of the Black Mingo Group is permeable and consists of sand interbedded with clay, limestone and sandstone. This portion of the unit is hydraulically connected to the Santee Limestone and therefore, considered part of the greater Floridian Aquifer system (Park, 1985; Petkewich, et al., 2004; Park, online report: NOAA-NERRS ACE Basin Characterization; Hockensmith, personal communication, 2012). Water from the Black Mingo aquifer system is soft and has high concentrations of bicarbonate. Salinity and fluoride content tends to increase locally with increased proximity to the coast (Park, 1985).

3.2.3. Eocene (Santee-Black Mingo) Floridian Aquifer

The Santee Limestone and the upper Black Mingo Group comprise the northernmost extension of the Florida Aquifer System (Figure B-7), which extends across South Carolina, Georgia, Alabama and into Florida (Parks, 1985; Miller, 1990; Petkewich et al., 2004; Hockensmith, personal communication, 2012). Within the Charleston area, the Floridian aquifer consists of carbonate and sandy strata belonging to the Moultrie Member of the Santee Limestone, and the upper 50-feet of the Black Mingo Group. The aquifer is confined by the Cross Member of the Santee Limestone and the Cooper Formation (Park, 1985; Petkewich et al., 2004). The aquifer is approximately 200 feet thick in the vicinity of Charleston, South Carolina, but gradually thickens to 3,400 feet beneath southern Florida (Miller, 1990). The top of this aquifer lies between -250 and -300 feet msl beneath Charleston, S.C. Wells drilled into this aquifer range in depth from 30 to 100 feet deep near Moncks Corner and Lake Moultrie, to 200 to 450 feet deep near south-central Charleston. The Santee Limestone contains zones of permeable limestone separated and confined by impermeable beds of limestone. Permeability is variable but is low compared to the underlying sandy strata of the Black Mingo Group. Therefore, wells are commonly drilled and screened to include both units for consistent water flow. Transmissity within the aquifer system ranges widely from 500 to 3700 ft^2/day and the hydraulic conductivity ranges 29 to 170 ft/day. Average water yield from established wells is up to 300 gpm. The Santee-Black Mingo Floridian aquifer reportedly provided sufficient volumes of groundwater for domestic residential use; however, over-pumping has resulted in long-term declines in water levels and, localized sink-hole activity (Park, 1985). Figure B-7 characterizes the effects that over pumping have on regional scale groundwater movement. Prior to extensive well drilling, groundwater generally moved southeast from upland recharge areas towards the coast. Drilling and development of the Floridian Aquifer System resulted in large potentiometric lows centered under large metropolitan areas such as Charleston, SC (Park, 1985; Miller, 1990).

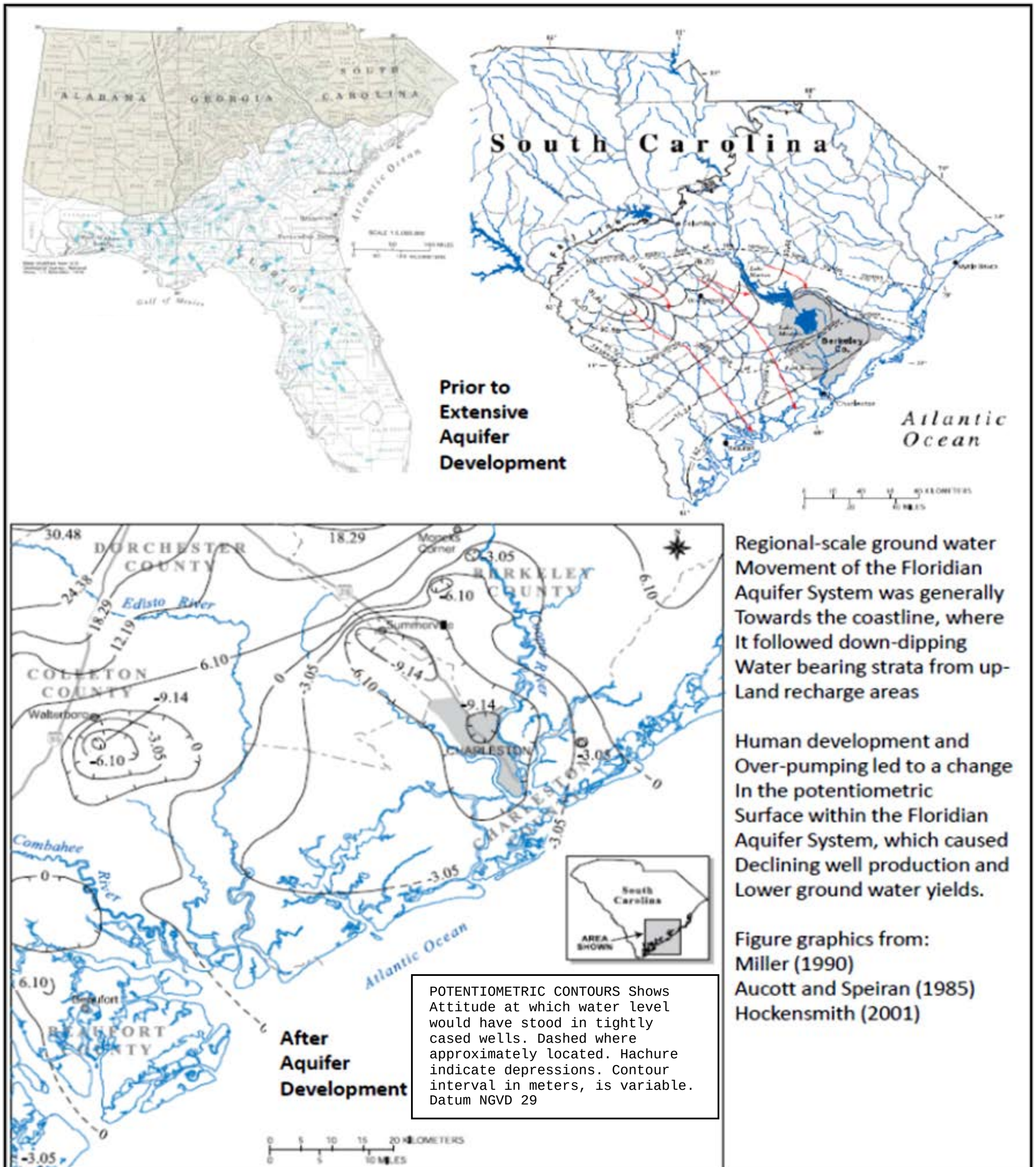

Figure B-7. Floridian aquifer system and potentiometric surface beneath Charleston, SC

3.2.4. Late Eocene-Oligocene Cooper Group Aquiclude

The Cooper Formation forms an impenetrable confining unit between the Santee-Black Mingo aquifer system and the overlying Quaternary unconfined surficial aquifer system. The thickness of the Cooper Formation ranges from 240 to 260 feet thick beneath Charleston (Park, 1985; Hockensmith, personal communication, 2012). The Cooper Formation has extremely low permeability and hydraulic conductivity, although localized zones of permeable material do exist. Park (1985) mentions the presence highly permeable limestones within the Cooper

Formation that occur at depths -200 to -500 feet msl beneath Edisto Island. Higher up within the unit, porous limestones (Park, 1985) and sand lenses (Brainard, et. al., 2009) occur at depths of -50 to -90 msl Brainard et. al. (2009) describes the presence of 30 foot thick sand lenses that were encountered in the Cooper Group during a recent tunnel expansion for the Charleston Water System beneath Daniel Island. These zones of high porosity strata are confined and generally of limited extent, therefore, they are not generally considered reliable sources of groundwater (Parks, 1985).

3.2.5. Quaternary Unconfined Surficial Aquifer

The surficial unconfined aquifer consists of all strata that are younger than those of the Cooper Group, which includes; the Ten-Mile Beds, Wando Formation and the Pleistocene-Holocene barrier complex deposits. The thickness of this aquifer ranges from 40 to 65 feet thick within the Charleston area. Groundwater occurs at water-table depth, which ranges from 3 to 15 feet below ground surface and fluctuates annually between 1 to 6 feet. Recharge is usually through local rainfall, although some water is contributed by the underlying Santee Limestone where the Cooper Formation is thin or absent. Groundwater from the surficial aquifer is acceptable for general use, but its yield is not consistent enough to be considered for widespread use. In addition, salt-water intrusion as a result of over-pumping, has limited the use of this aquifer for municipal use (Park, 1985). Wells drilled into this aquifer mainly serve limited residential and irrigation use (Hockensmith and Doars, personal communication, 2012).

3.3 Inventory of Existing Water Resources

3.3.1. Charleston Water System

Historically, the City of Charleston relied upon shallow wells and collected rainwater to supply the drinking water needs during the Colonial Era. As the population grew, the need for a clean, safe potable water source became apparent; therefore, the city commissioned the drilling several deep wells to supply drinking water to the city's population. From 1823 to 1879, several attempts were made to drill to deep wells to tap into the deeper confined aquifer, which were more desirable in terms of water quality, yield, and sanitation. The first producing municipal well was completed in 1879 to a depth of 1,970 feet and had a yield of 486 gpm. Continual growth of the port city rapidly outpaced the drilled aquifer water supply, and so the City of Charleston commissioned the construction of dams to impound Goose Creek to provide a more reliable water supply (http://www.charlestonwater.com/water_history_part1.htm, accessed 27FEB12).

Figure B-8. Charleston Water System service

Today, the main provider of drinking water to the greater Charleston Area is the Charleston Water System. The Charleston Water System was first established in 1917 and now serves over

400,000 people in the municipalities of Charleston, North Charleston, West Ashley and surrounding areas (Figure B-8). The Charleston Water System draws it water from two sources; the Bushy Park Reservoir and from the Edisto River, near Givhans Ferry. Water from these two sources is piped to the Hanahan Water Treatment Plant, which has a processing capacity of 118 million gallons per day. Once treated, the water is transferred into the water distribution system which consists of 1,600 miles of water mains. The Charleston Water System is presently replacing a network of tunnels that carry sewage to the Plum Island Treatment plant. This project was estimated to cost 224.5 million, and it is presently in phase 5 of 6 in order of completion (http://www.charlestonwater.com/water_history_part2.htm, accessed 07FEB12).

3.3.2. Water Wells within Charleston County

Well data for Charleston County was accessed from the South Carolina Department of Natural Resources Hydrology Section website (http://www.dnr.sc.gov/water/hydro/data.html) and was plotted in ArcGIS in order assess the depth and proximity of well borings within Charleston County. These were then sorted according to depth into shallow (0-60 feet) and deep (>60 feet).

There are presently approximately 676 registered water wells within Charleston County (Figure B-9). Figure B-10 illustrates the primary distribution of uses for these wells; 1) domestic consumption (33%), 2) irrigation (11%), public sector (9%), and industrial (3%). A percentile of these wells are no longer usable (9%), have been abandoned (6%) or are designated for observation and monitoring (8%) purposes. Drilled wells that have the greatest groundwater yields are used for commercial/private irrigation, public sector, and industrial purposes (Figure B-11), which are drilled to greater depths than conventional wells drilled for domestic consumption (Figure B-12). These deep wells are drilled and cased to draw from several water bearing zones throughout the Eocene Floridian (Santee-Black Mingo) aquifer system, which provides the most consistent and highest-quality water supply. The cost to drill these deep wells is prohibitive to most users, who have often opted to only drill into the upper Santee Limestone.

Shallow wells set into the Quaternary aquifer system (< 60-feet deep) comprise approximately 28% (189) of the total (676) number of wells drilled within Charleston County. Of these shallow wells, approximately 31% (59) are used for domestic use, 12% (22) for irrigation, and 6% (12) are designated for public use. Unusable and abandoned wells (35) comprise an additional 19%. The remaining wells (26%) are designated for testing and observation only. Production yields reportedly range from 0 to 200 gpm, with 50 gpm being most common on active wells.

Figure B-13 shows the location of shallow wells within the vicinity of the harbor project site. These wells are drilled down to -60 feet depth msl and have varying screened intervals 0 and -45 feet msl in order to intercept the water table. The SCDNR well registry data indicates that many of the wells are presently unusable or abandoned. Those that are in use generally have low production yields (< 25 gpm) and are used for only for irrigation or domestic purposes (Figure B-14). Municipal wells owned by the Town of Mount Pleasant have larger yields of 150-200 gpm, but they are not used due to contamination, saltwater intrusion, or decommissioning.

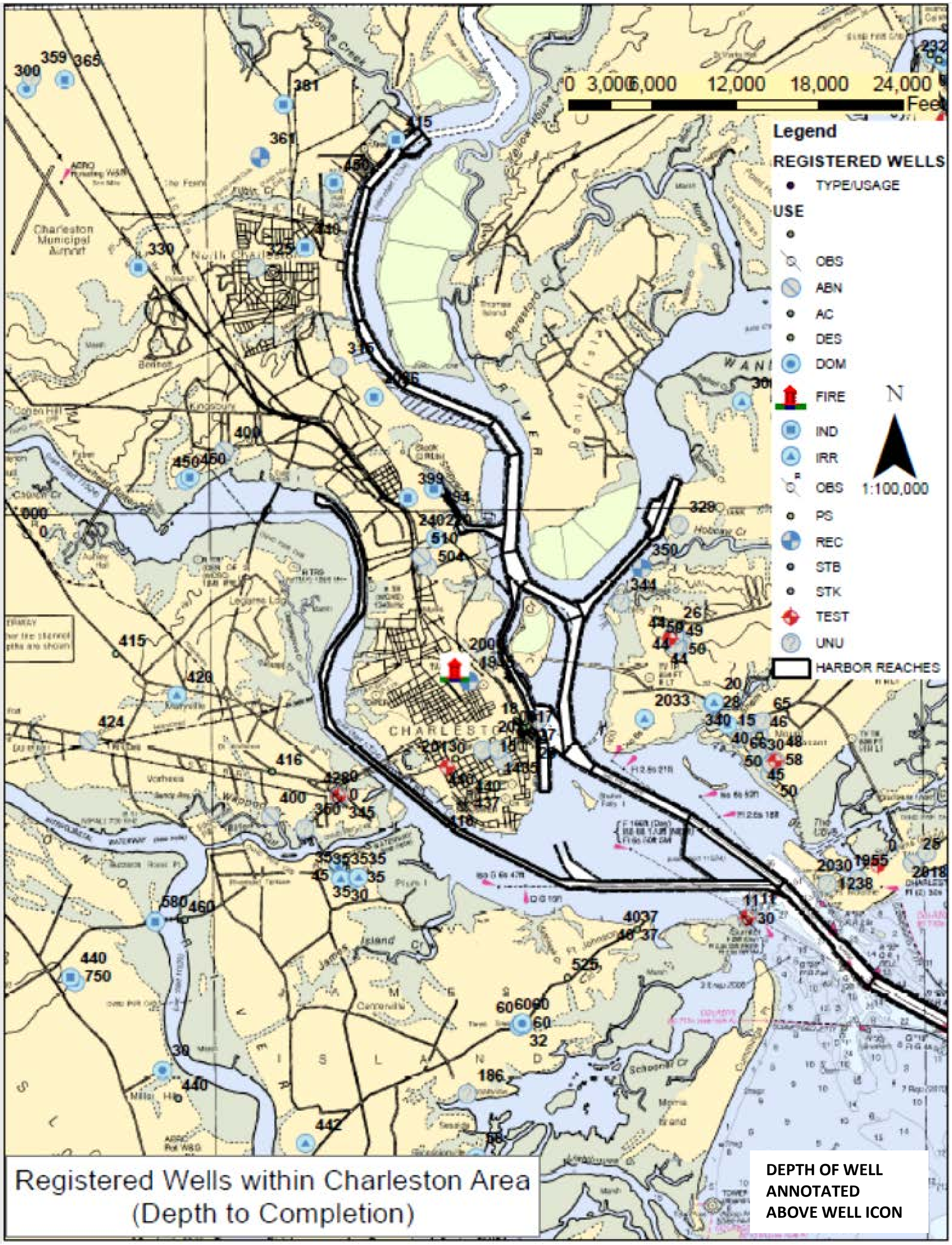

Figure B-9. Map of wells registered with SCDNR in Charleston, S.C.

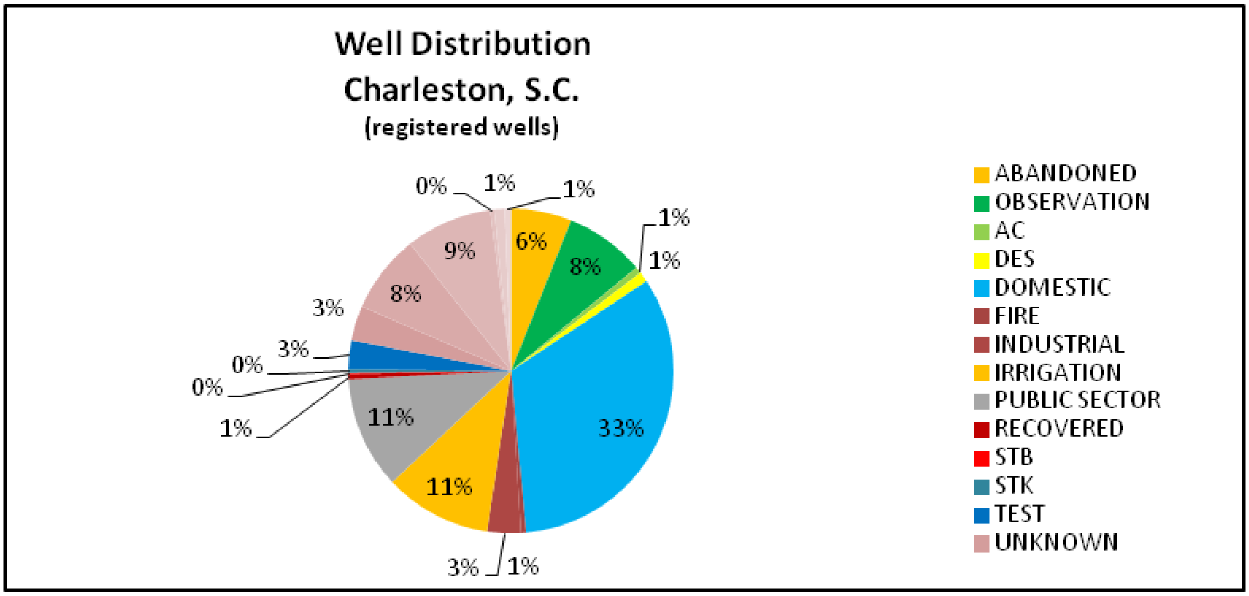

Figure B-10. Distribution of registered well types in Charleston, S.C.

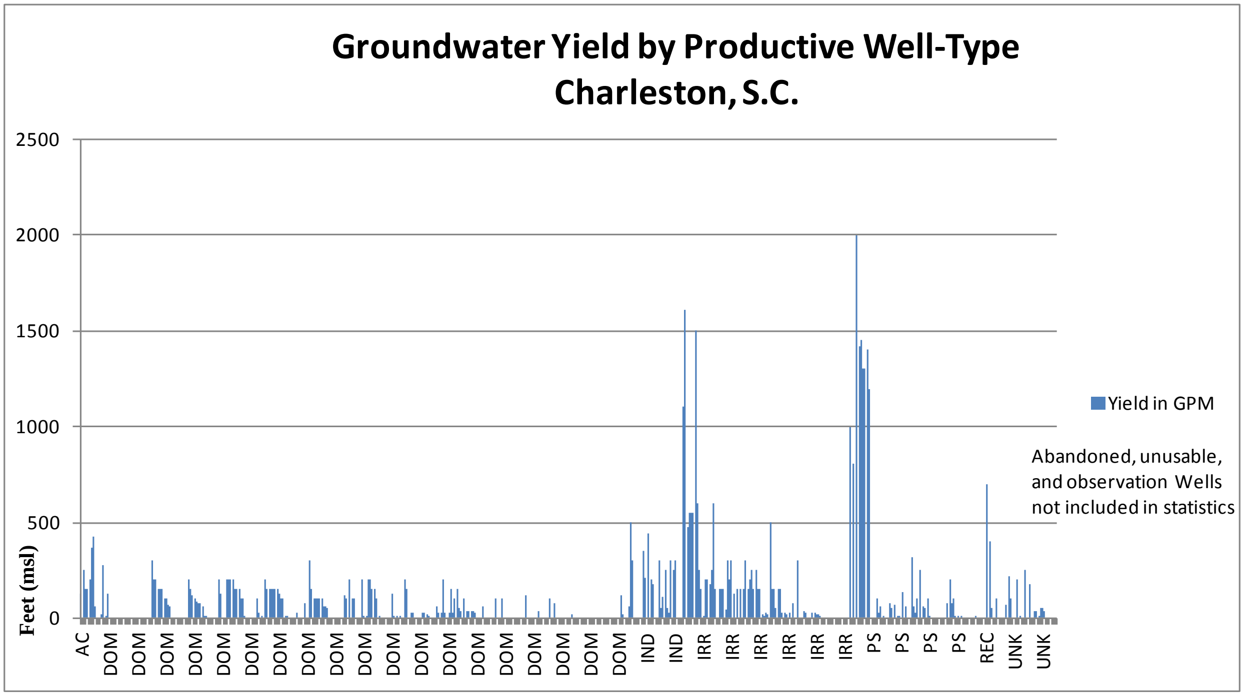

Figure B-11. Groundwater yield by major well type in Charleston, S.C.

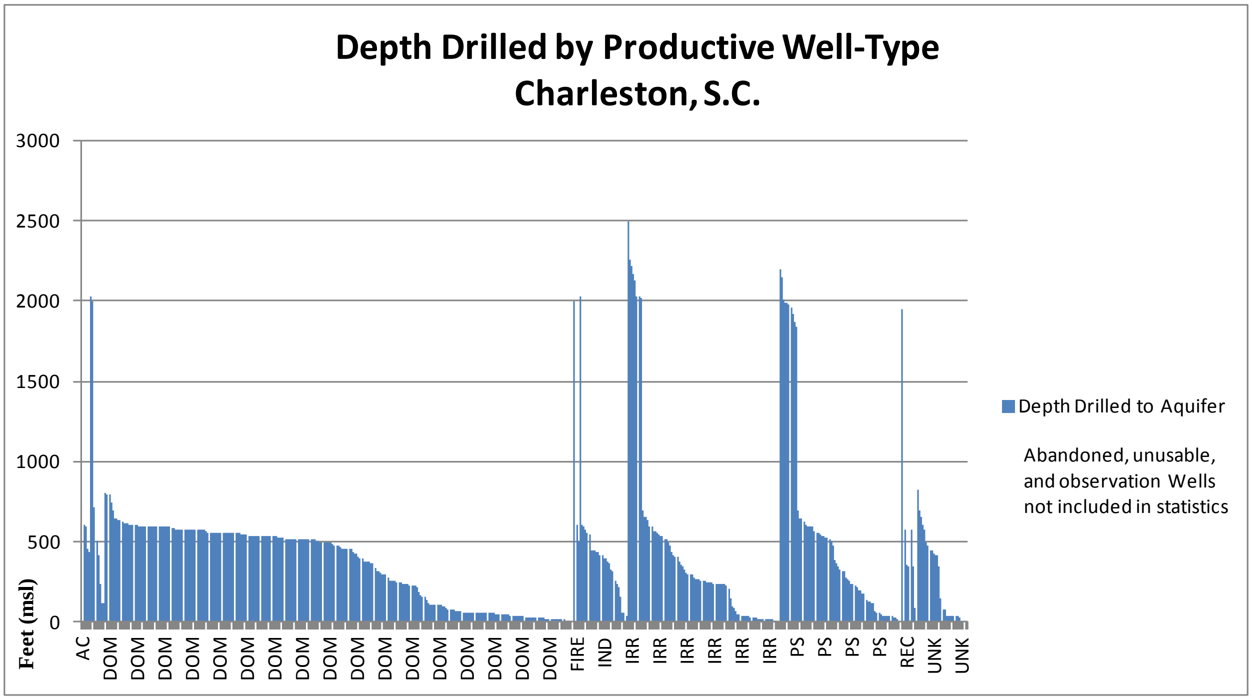

Figure B-12. Depth of all major producing water wells in Charleston, S.C.

The majority of the wells within the Charleston Area are drilled much deeper than the -60 foot MLLW threshold established for this evaluation. These wells are drilled to depths ranging from 160 to 2,200 feet deep and have screened intervals starting from -220 to -1,850 feet msl. Of the 487 deep wells, approximately 40% (195) are for domestic use, 12% (60) are designated for irrigation, 11% (56) reserved for fire and public sector consumptions, with lesser percentages designated to industrial (5%), observation and testing (4%), and other (4%). Abandoned and unusable wells collectively (73) account for 15% of the deep wells, while the remaining 9% have an unknown status. Table 4 shows that domestic-use wells, which comprise the majority of deep wells, are generally drilled to depths around -500 feet msl. This was done in order to tap into the upper-mid water bearing zones of the Santee Limestone. Generally, groundwater yield increases by orders of magnitude with depth drilled. Data presented in Figure B-12 and Figure B-16 indicates that public sector and irrigation wells are drilled much deeper into the Floridian aquifer than domestic/residential users. These wells are screened in such a way as to draw from several water-bearing zones within both the Santee and the Black Mingo Group in order to draw greater and more consistent yields (Figure B-18).

Figure B-15 shows the deep water wells adjacent to the project and their screened interval. The wells are predominantly designated for industrial, commercial, irrigation, and public sector use. The well screens are set much deeper (60-1,200 feet below ground surface) than the -60 feet MLLW elevation threshold that is established for this evaluation. Figure B-17 shows that deep wells located in Isle of Palms and Mount Pleasant have historically the greatest water yields, but several have been abandoned for unknown reasons as shown in Figure B-15. At least two wells are still active on Isle of Palms that have yields ranging from 500-1500 gpm. A handful of deep active wells having yields ranging from 500-1500 gpm are present in Mount Pleasant and are designated for public sector and irrigation use. Within Charleston and North Charleston, many of the producing deep wells (-200 to -500 feet msl) are designated for industrial use and have yields ranging from 200 to 500 gpm (Figure B-15 and Figure B-17).

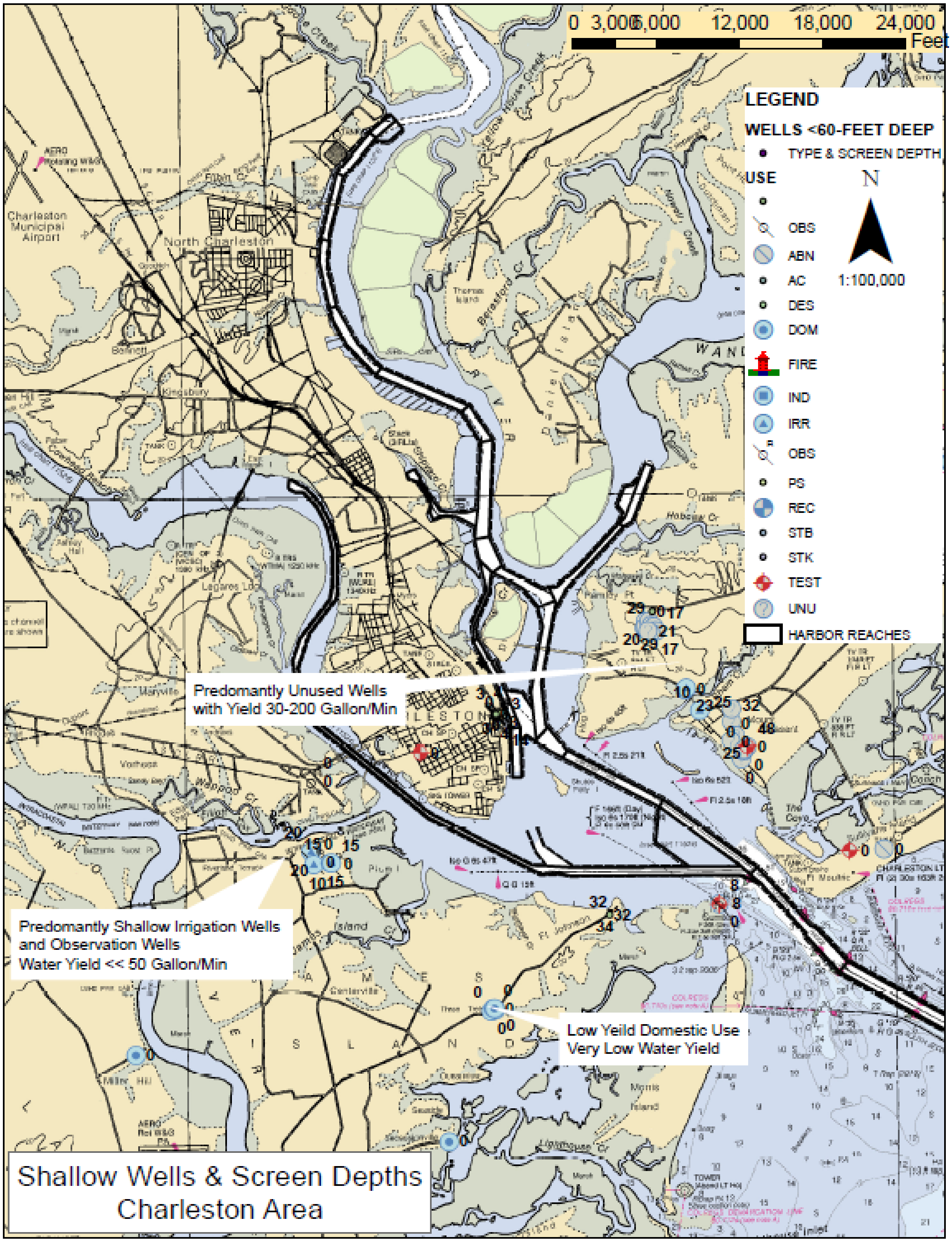

Figure B-13. Map of wells adjacent to Charleston Harbor that are less than 60 feet deep.

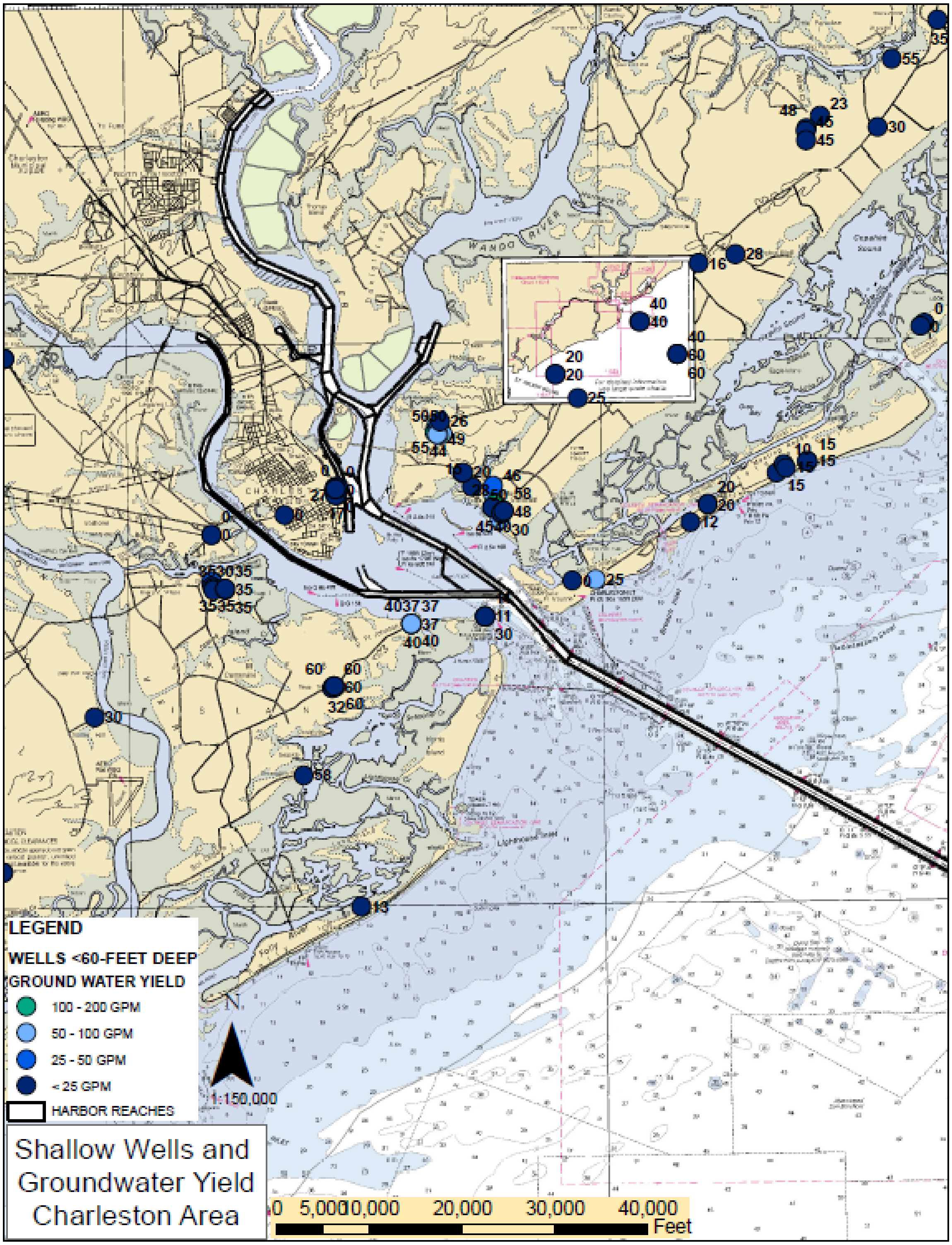

Figure B-14. Map showing groundwater yield from wells drilled into the surficial aquifer.

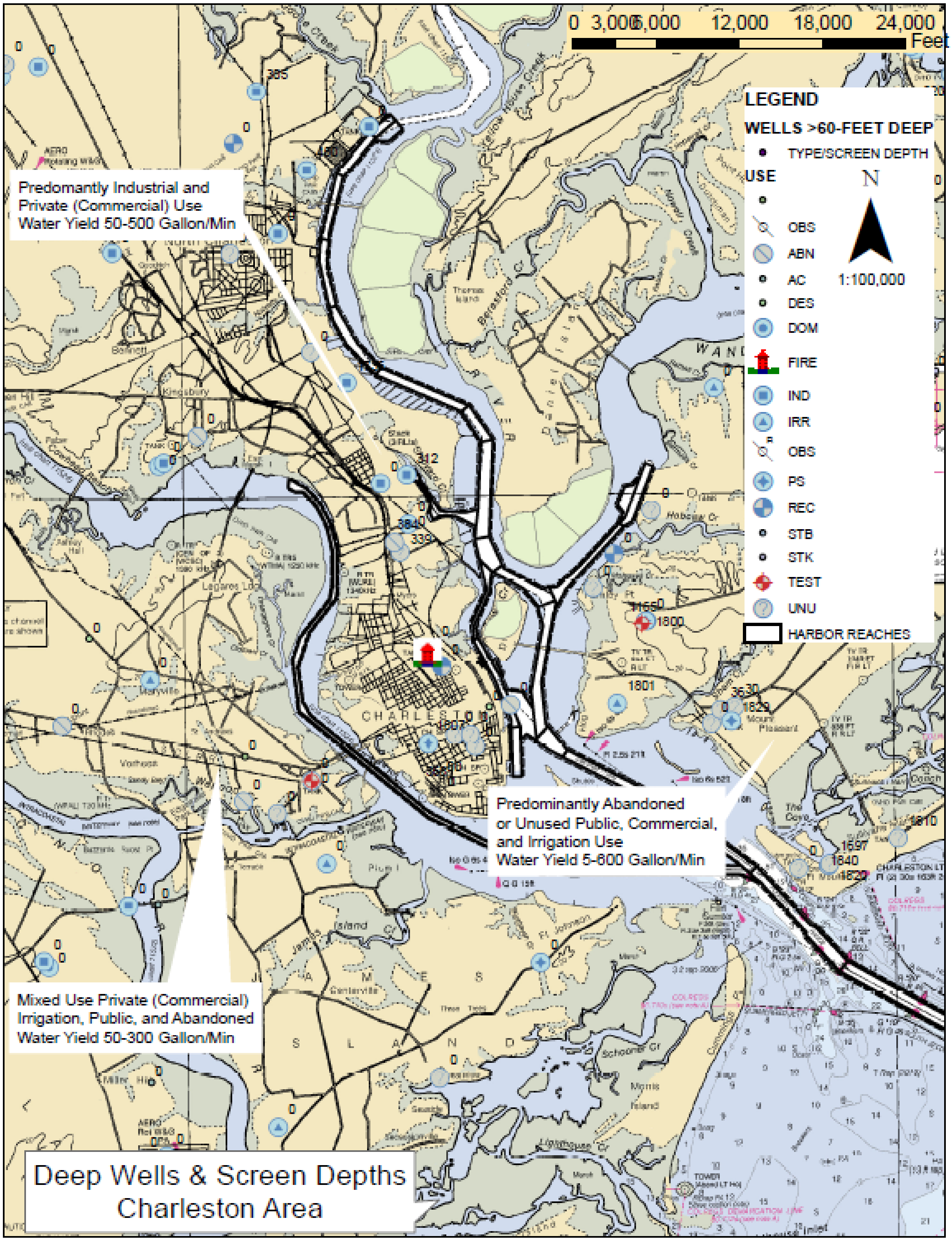

Figure B-15. Map of wells adjacent to Charleston Harbor that are greater than 60 deep.

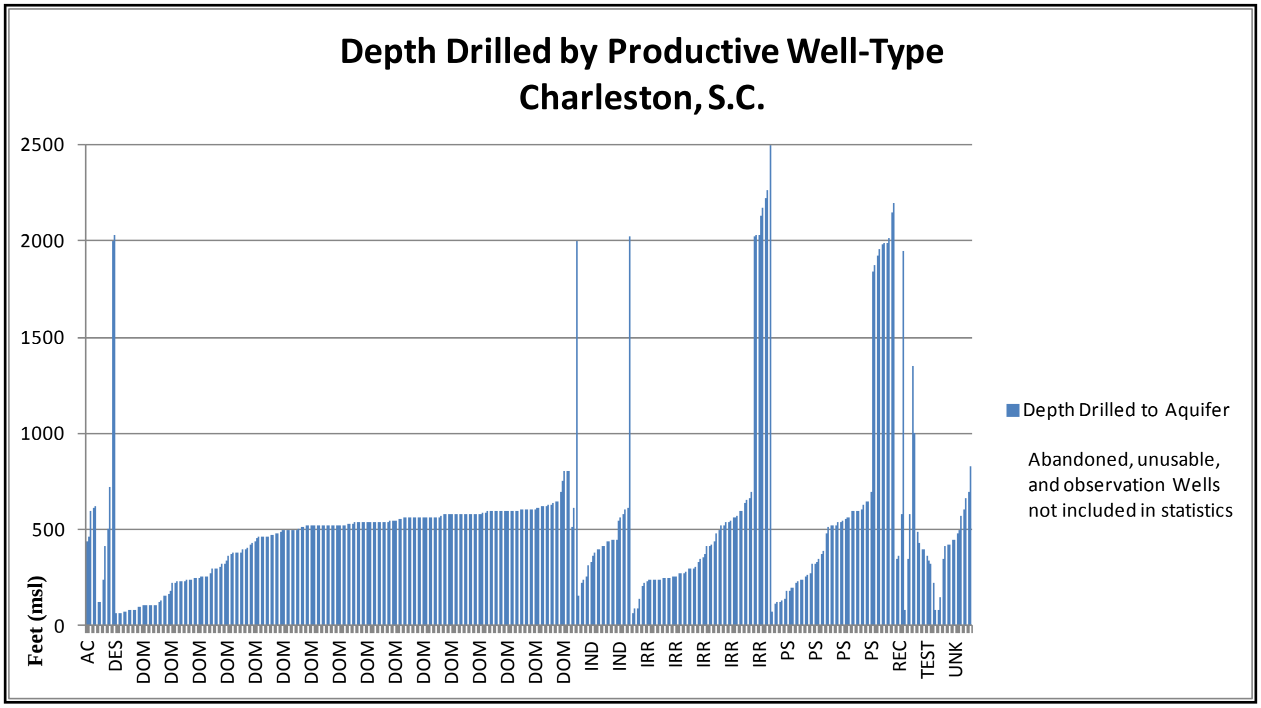

Figure B-16. Chart of deep well types in Charleston, S.C.

The color coded distribution of well yields in Figure B-17 does not indicate that there is any apparent relationship between the productivity of a well and its proximity to the harbor. Rather, as depicted in Figure B-16, deeply drilled public sector and irrigation wells (depicted in Figure B-17, Figure B-12, and Figure B-16) have the greatest yields. The groundwater yield of the majority of the wells in Charleston is controlled by the depth of well and its screened interval; not proximity to the harbor project. These high production wells tap aquifers that are effectively isolated by their relative depth.

3.4. Aquifer Sensitivity to Channel Deepening

3.4.1. Existing Harbor Dredge Prism

The presently maintained channel depths within Charleston Harbor is -45 feet within the upper and lower portions of the harbor, and -47 feet within the entrance channel (Figure B-19). All dredged channel elevations are referenced to Mean Lower Low Water (MLLW). The present maintenance dredging includes a 2-foot over-depth allowance for dredgeability. The proposed deepening project could deepen the entrance channel to a maximum of -58 feet MLLW and the upper and lower portions of the harbor to -52 and -56 feet MLLW. These proposed design depths include over-depth and advanced maintenance provisions. Presently, the frequency of maintenance dredging is scheduled once every 18-21 months for the upper harbor, 12-15 months for the lower harbor and entrance channel.

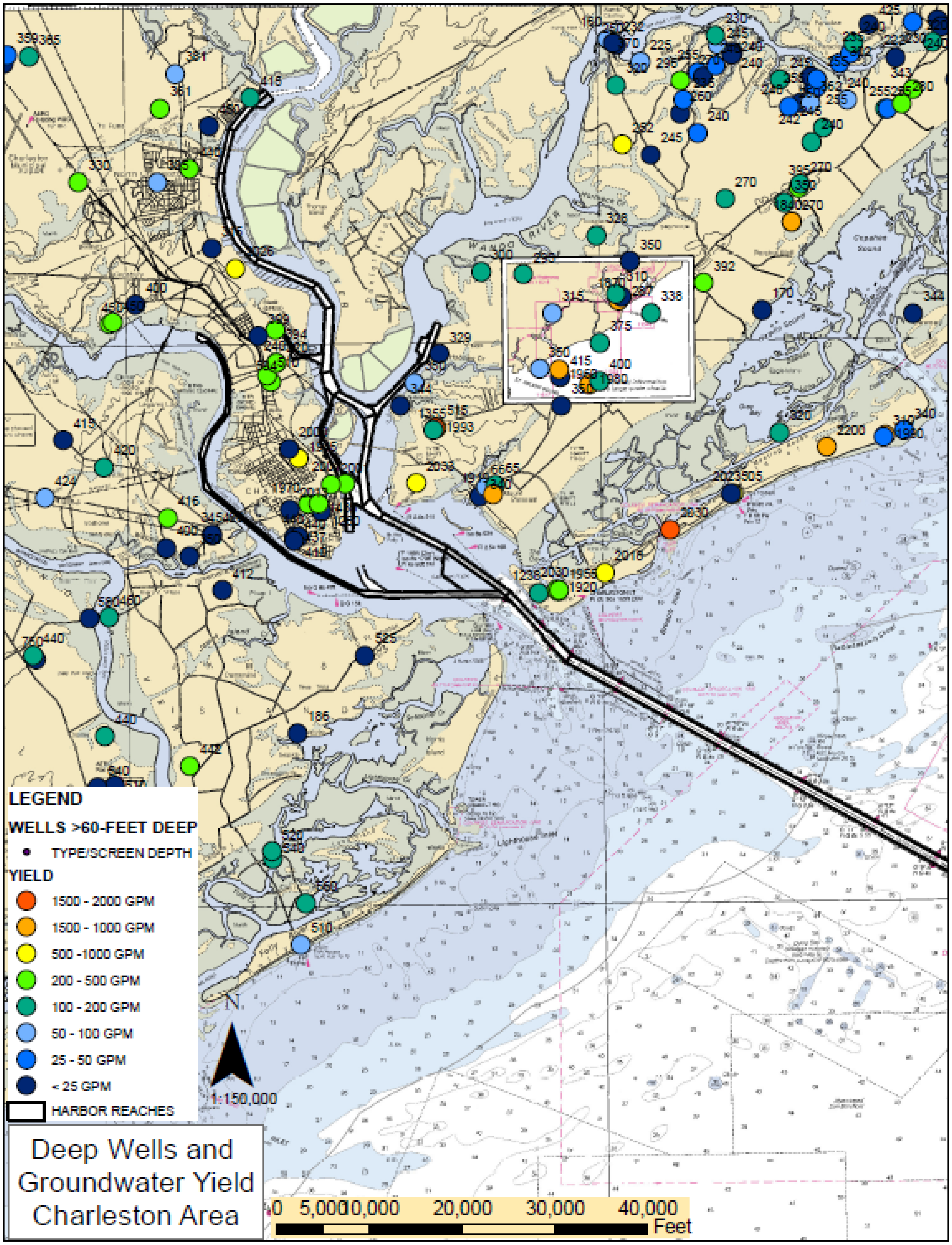

Figure B-17. Map of groundwater yield in wells drilled into the Floridian aquifer.

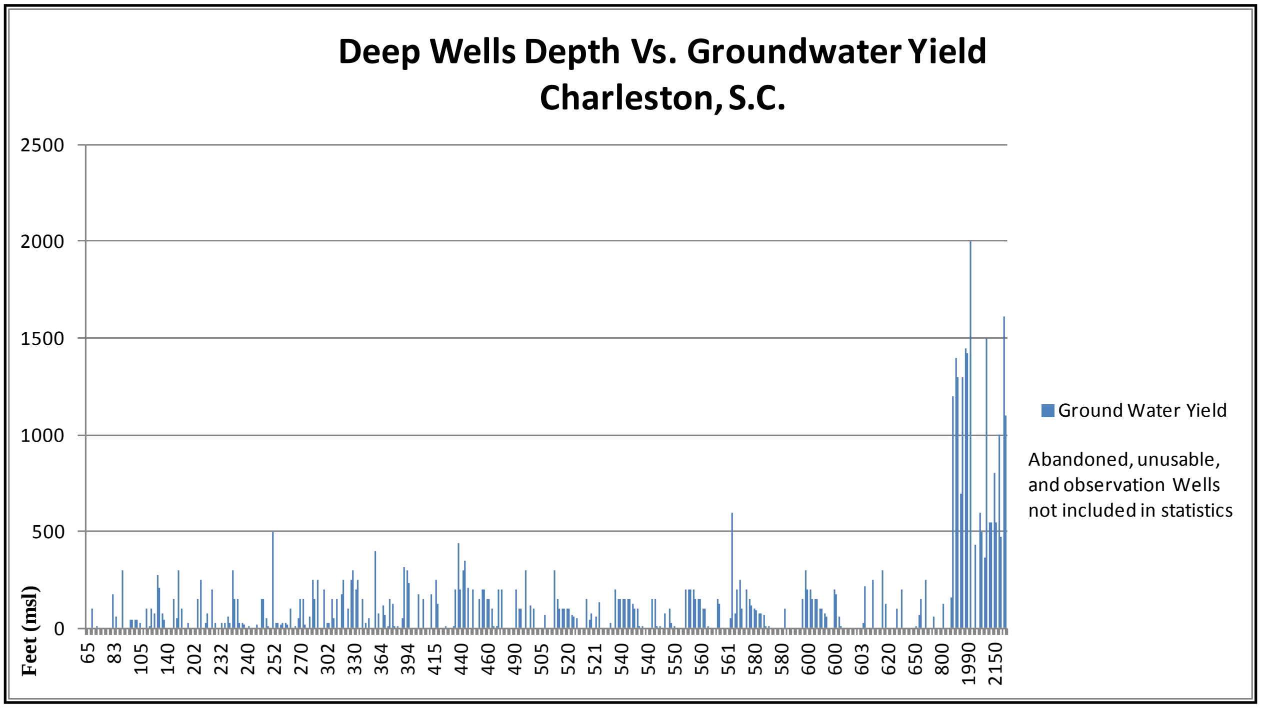

Figure B-18. Chart of groundwater yield by well depth.

3.4.2. Strata within Proposed Harbor Deepening

Stratigraphic units that are most likely to be encountered during the proposed deepening are 1) the Goose Creek Limestone; 2) the Marks Head Formation; 3) the Edisto Formation; and 4) the Cooper Formation. The Goose Creek Limestone and Marks Head Formation beneath Charleston area may be encountered at elevations ranging from -40 to -80 msl near the mouth of the harbor (Weems and Lewis, 2002). Weems and Lemon (1993) mapped these units as discontinuous strata from boring logs; however their cross-sections indicate that the Marks Head underlies mouth of the harbor (Figure B-20). Seismic profiles of Harris et al. (2005) indicate the presence of the Marks Head Formation south of Charleston Harbor at elevations -30 to -60 feet msl near Folly Island. The Edisto Formation lies stratigraphically between the Marks Head Formation and the Cooper Formation (Weems and Lemon, 1993). Ward et al (1979) apply the term “Edisto Formation” to encompass sandy limestones of Miocene age that unconformably overlie the Cooper Formation. This unit occurs as surficial thin erosional outliers northwest of Charleston, near Summerville, S.C., and in the subsurface beneath Charleston at elevations -10 to -20 msl based upon boring data (Weems and Lewis, 2002).

The Cooper Formation underlies the aforementioned strata and is significantly thicker and more widespread. Figure B-20 shows that the unit generally dips southeast, extending seaward with depth (Park, 1985; Weems and Lemon, 1993; Weems and Lewis, 2002). The unit is generally described as a thick, impermeable, confining unit that is composed of clayey to silty limestones and stiff calcareous silty-clay and clayey silts (Park, 1985, Weems and Lemon, 1993; Petkewich, et al., 2004; Brainard, et al., 2009; Hockensmith and Doars, personal communication). It has very poor water conductivity due to its high impermeability. The Cooper Formation floors the upper and lower harbor based upon historical boring data. The elevation to the top of the Cooper

Formation ranges from -39 feet near the southern tip of Daniel Island, and deepens both to the southeast and northwest to -52 feet msl. The unit is estimated to be at least 240 to 260 feet thick (Park, 1985). The top of Cooper Formation is believed to gently dip and thicken toward the southeast (USACE-SAC, 2002; Park, 1985; Weems and Lemon, 1993). Based upon the existing data the Cooper Formation may extend below the continental shelf, except where exposed by erosional escarpments and channeling activities.

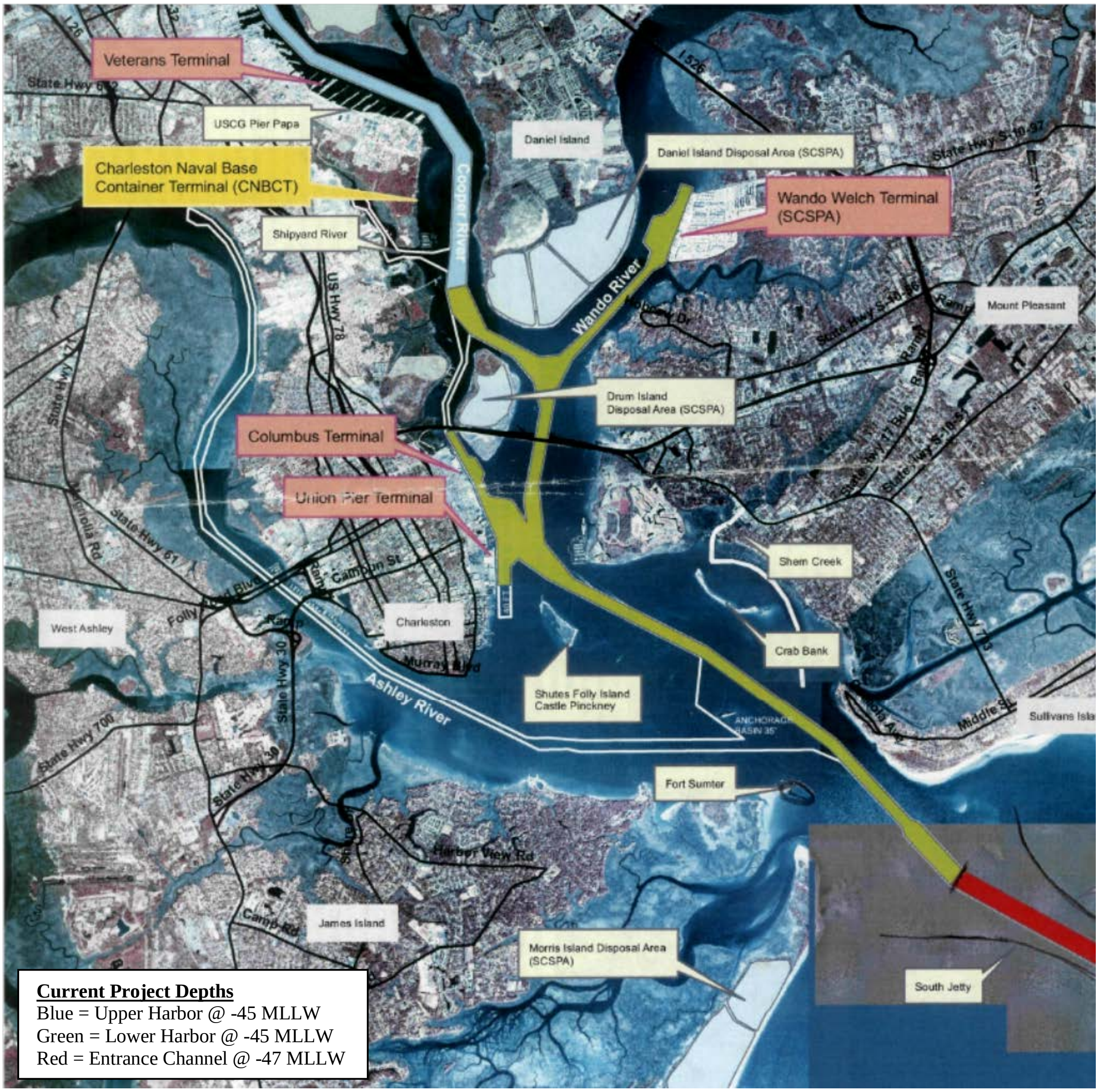

Figure B-19. Charleston Harbor channel reaches.

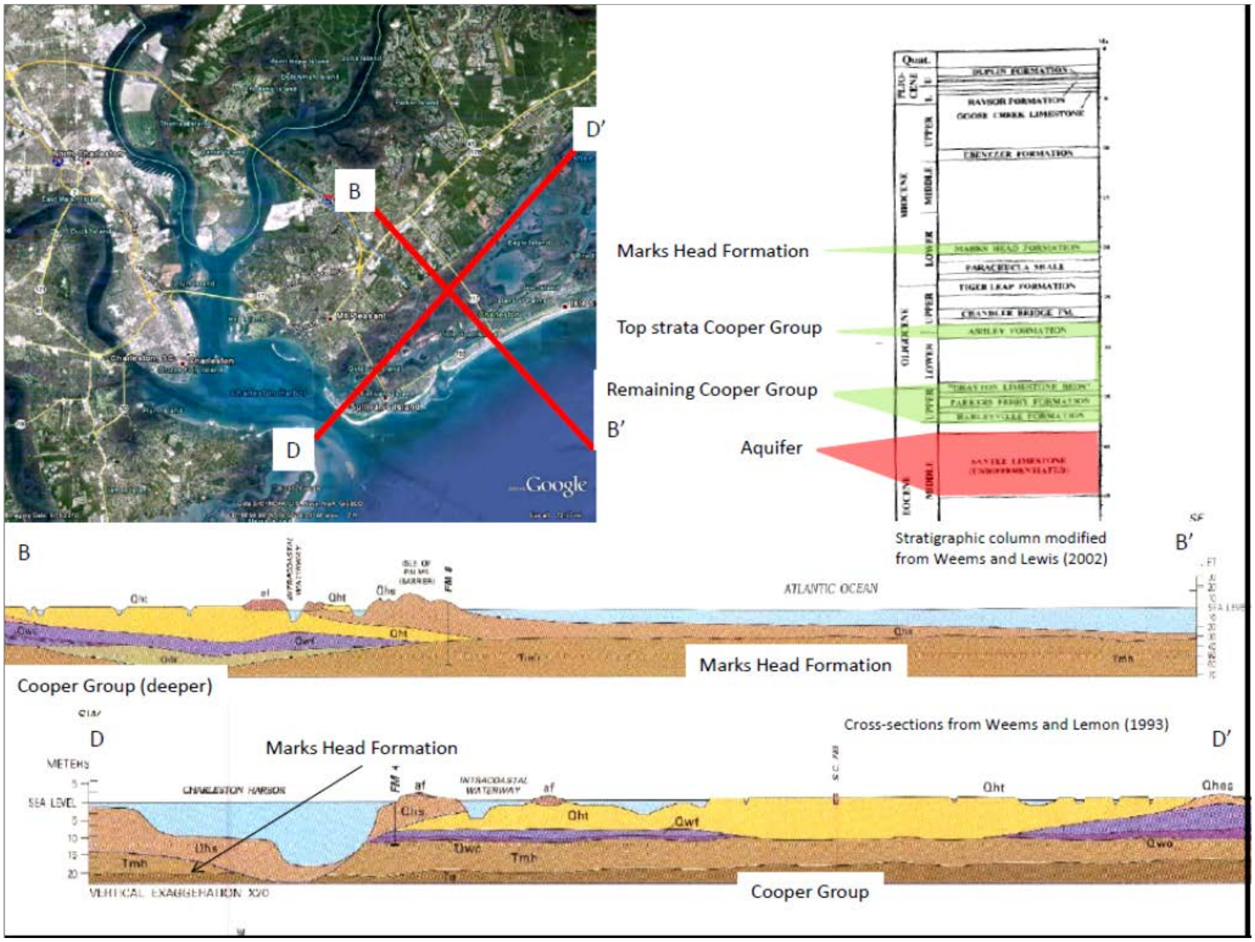

Figure B-20. Geologic cross-sections of Charleston quadrangle, modified from Weems and Lemon (1993).

Dredging activities associated with deepening of the harbor will intersect the Cooper Group, portions of the Edisto Formation, and possibly some of the surficial Quaternary deposits. There are no hydrologic concerns for dredging into these units. The primary Floridian aquifer will not be encountered. Dredging a deeper channel into the Cooper Formation may expose occasional sand horizons and perched water tables; however, these are limited in extent and are not used for water resources (Park, 1985; Brainard, et. al., 2009). The unit is sufficiently thick enough to effectively isolate the underlying Floridian aquifer (Santee Limestone-Black Mingo Group) from dredging activities. If a 60-foot deep buffer zone were extended across the entire harbor project, the deepest stratum intersected would be the uppermost strata of the Cooper Formation (Figure B-20). This material is generally described as a “marl” consisting of weakly cemented, calcareous, silt-clayey fine sand and sandy silt. This stratum is estimated to be 125-133 feet thick and it sits atop an additional 115-127 feet of impermeable material (Park, 1985; Weems and Lemon, 1993). Risk of breaching the underlying Floridian aquifer is essentially non-existent with the proposed project dredge prism.

3.4.3. Proposed Deepening and the Floridian Aquifer

The Floridian Aquifer system as discussed earlier consists of portions of the Santee Limestone and Black Mingo Group, the top of which is lies -200 and greater below the surface beneath the city of Charleston. The deeper Cretaceous Cape Fear, Late Cretaceous Middendorf and Black Creek aquifers are several hundred to several thousand feet deep, sufficiently confined, and not widely developed; therefore they are of no further concern for this assessment. Referring to the cross-sections (Figure B-20) of Weems and Lemon (1993) and the top of Santee Limestone map (Figure B-3) of Park (1985), it is clearly evident that this aquifer is well below the dredging depth of the proposed deepening.

3.4.4. Previous SCDNR Groundwater Impact Statement (1995)

The South Carolina Department of Natural Resources was consulted in 1994-1995 by the Charleston District for insight on potential dredging impacts groundwater during the previous deepening project. The SCDNR Hydrology Department provided a memorandum for record stating no adverse impact to the Floridian Aquifer System, if the channel were deepened to -45 feet MLW (see Figure B-21). The reason for this decision was the great thickness of the Cooper Group that overlies the aquifer. The thickness of this stratum is stated in the document to be 200 to 260 feet thick beneath the project site.

South Carolina Department of
Natural Resources

James A. Timmerman, Jr., Ph.D.
Director
Alfred H. Vang
Deputy Director for
Water Resources

February 6, 1995

Ms. Robin Socha
EN-PR
Dept. of the Army
Charleston District, Corps of Engineers
P.O. Box 919
Charleston, SC 29402-0919

RE: Charleston Harbor Deepening Project

Dear Robin,

I have reviewed the 404(b)(1) Evaluation for the Charleston Harbor Deepening Project for any potential adverse impacts on underlying aquifers. The project involves deepening the Charleston Harbor from 40 feet to between 42 and 45 feet below mean low water.

According to SCDNR-WRD records, the top of the Cooper Formation lies between the approximate elevations of -10 and -60 feet mean sea level in the project area, with thickness varying from 200 to 260 feet. This formation acts as the upper confining layer to the Santee Limestone. The aquifers of the Santee Limestone and the underlying Black Mingo Formation contain salt water in the vicinity of Charleston Harbor.

In light of hydrogeologic conditions, no adverse impacts to aquifers are expected as a result of deepening Charleston Harbor by a maximum of five feet. Should you need additional information, please feel free to contact this office.

Sincerely,

Brenda L. Hockensmith P.G.

Brenda L. Hockensmith, P.G.
Senior Hydrologist

cc: Rod Cherry, Section Chief
A. Drennan Park, Regional Hydrologist
file

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Figure B-21. Statement of No-Impact for previous harbor deepening from SCDNR, Hydrology Section.

3.4.5. Impact on Quaternary Aquifers

The surficial aquifer is found within the upper 65 feet of the subsurface and is tied to the water table. This aquifer and the wells drilled into already lies within the depth prism of the existing project, and no losses relating to previous dredging have been established. The surficial aquifer is not horizontally continuous or has a constant thickness because the Quaternary strata in which they are perched consist of unconsolidated sands and interbedded clay. Water yields are generally low. Because there is no confining layer, the potentiometric surface follows the water table, which flows down slope following the local topography. In addition, these aquifers are sensitive to drought-induced water-level fluctuations and salt-water intrusion by virtue of their proximity to the coast. Over-pumping has led to saltwater intrusion in municipalities of Folly Beach, Mt. Pleasant, Fort Sumter, and Porches Bluff (Park, 1985), prior to the harbor deepening activities in 1995. Presently, there are few wells tapped to the surficial aquifer system that are used for domestic consumption in the Charleston area; therefore, very little impact is anticipated with the proposed channel deepening.

3.5. Groundwater Assessment Conclusions

Based upon the geologic setting, depth and thickness of the local stratigraphy, there is no impact anticipated to the Floridian Aquifer System, as a result of the proposed Charleston Harbor deepening. The Floridian Aquifer System is effectively isolated from any dredging activity by a thick (200-260 ft) sequence of impermeable strata. Furthermore, this strata and the Floridian Aquifer System dips and thickens seaward to the southeast, which further isolates it from the relatively shallow dredging.

There is little to no impact anticipated to the shallow surficial aquifer system. Much of this aquifer system already lies within the depth prism of the present project, and no problems relating to the 1995 harbor deepening have been reported. Because these aquifers are not confined and are prone to drought-related fluctuation, they are not considered consistent sources of water. In addition, many of the shallow wells in close proximity to Charleston Harbor have already been designated unusable or abandoned due to saltwater intrusion. The leading cause for saltwater intrusion in the shallow aquifer system is population growth and overuse by residential irrigation systems, not dredging activities.

IV. SUBSURFACE INVESTIGATIONS UPPER & LOWER HARBOR

4.1 General

The subsurface conditions for the upper and lower harbor reaches are described this chapter in order to assess the feasibility of deepening from the present authorized depth of -45 feet to a maximum of -52 feet MLLW in the upper harbor, and -56 MLLW in the lower harbor. In areas where advanced dredging is authorized[7], the maximum dredge prism in the upper harbor may extend to -53 MLLW. Critical aspects to this assessment involve characterizing the soils, and/or presence of bedrock within the proposed dredge prism for both upper and lower harbor reaches. The entrance channel will be discussed separately in Chapter VI. A total of 406 borings spanning from 1972 to 2004 were screened and analyzed in order to accomplish this task. No new geotechnical drilling in the upper and lower harbor was conducted for feasibility study. Existing historical data was post processed using gINT[8] geotechnical software and 3-D subsurface fence diagrams were generated in order to develop a subsurface model. This model provides an indicator to the lateral and vertical variability of materials present and facilitates cost estimation of dredging activities.

4.1.1. Purpose and Scope

The subsurface conditions within the upper and lower harbor reaches of Charleston Harbor will be delineated to elevations -52 and -56 MLLW in this chapter. The purpose is to verify material dredgeability and facilitate cost estimation of dredging activities. The primary data source used in this delineation will be historical borings that were input into gINT geotechnical database software. No new exploratory drilling in the upper and lower harbor reaches was conducted during the feasibility study. Three-dimensional subsurface fence diagrams are presented in Section 4.4. A description of the soils and stratigraphic conditions present for the upper and lower reaches are described in Section 4.5.

4.1.2. Upper & Lower Harbor New Work Removal Estimates

Initial volume estimates for material removal is presented in Table B-1. The material volumes are assumed to consist of unconsolidated sediment and that no bedrock is present.

Table B-1. Initial volume estimates for new work deepening, dated November 12, 2012.

Reach	Start Station	End Station	-52’ MLLW
Mount Pleasant Reach	900+00	995+18	215,472
Rebellion Reach	995+18	1077+91	364,979
Bennis Reach	1077+91	1155+87	405,921
Horse Reach	1155+87	1179+00	93,525
Hog Island Reach	1178+23	1273+12	635,334
Wando River Lower Reach	0+00	71+49	577,510
Wando River Upper Reach	71+49	119+78	301,307

[7] High shoaling areas in Lower Wando, Lower Town Creek, Ordnance Reaches, Ordnance Turning Basin, and Wando Turning Basin are required to have 45’ depth with 4’ of authorized advanced maintenance dredging and an additional 2’ allowable overdepth. Drum Island Reach is required to have 45’, plus 6’ of authorized advanced maintenance, and an additional 2’ allowable overdepth.
[8] gINT® is a registered trademark of gINT Software, which is owned by Bentley Systems, Incorporated.

Reach	Start Station	End Station	-52' MLLW
Wando River Turning Basin	70+76	109+00	220,202
Drum Island Reach	1273+12	1317+21	329,955
Myers Bend	1317+21	1342+77	249,338
Daniel Island Reach	1342+77	1412+71	729,106
Daniel Island Bend	1412+71	1440+86	160,853
Clouter Creek Reach	1140+86	1509+00	460,421
Navy Yard Reach	1509+00	1566+65	434,186
North Charleston Reach	1566+65	1615+95	272,571
Fiblin Creek Reach	1615+95	1664+72	178,318
Port Terminal Reach	1664+72	1701+00	215,396
Ordnance Reach	1701+00	1720+83	178,265
Ordnance Reach Turning Basin	1698+65	1720+83	488,345

4.2 Previous Supporting Investigations

There has been numerous geotechnical exploration programs conducted within Charleston Harbor since 1957. Drilling records prior to 1972 were unavailable for review and are presumably lost. There are 406 historical borings on record that have been drilled within the upper and lower harbor since 1972. Table B-2 and Table B-3 were created to catalogue the various subsurface efforts that have been completed for both upper and lower harbor reaches. The type of borings, general penetration depths, and number of borings that penetrate into the proposed dredge prism are discussed herein. Review of the existing drilling records indicates that the majority of borings sampled material that was already removed by previous dredging projects. Present project depths for the reaches range from -45 feet to -47 feet MLLW; however, many of the borings were drilled from 1972 to 1994 prior significant deepening and widening. Vibracore drilling was conducted throughout the harbor prior to the harbor being deepened to its present depth; however none penetrate to the proposed project depth of -52 feet MLLW.

4.2.1. Upper Harbor Borings

There have been a total of 251 Standard Penetration Test[9] (SPT) and vibracore borings drilled in the upper harbor by USACE, Savannah Core Drill Unit, Athena Technologies, Inc., and General Engineering, Inc., from 1988 to 2009. The maximum penetration depth of these borings ranges from -28 feet to -79 feet MLLW. Of the 251 borings drilled, only 94 penetrate to the maximum proposed dredging depth of -52 feet MLLW (Table B-2). A review of the pertinent drilling logs indicates that no rock was encountered.

[9] The Standard Penetrometer Test (SPT) is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil. The test involves driving a split-barrel sampler, a standard distance, using a standard weight and energy, in order to measure the penetration resistance of the soil, and recover samples for identification and lab testing. The SPT method described in detail by ASTM D 1586-84.

Table B-2. Catalogue of exploration drilling within the upper harbor reaches, Charleston Harbor.

Reach	Year	Driller	Boring ID	Total # Borings	Boring Type[10]	#Drilled to -45 ft MLLW	#Drilled to -52 ft MLLW	Max depth drilled	Rock Sampled?
Shipyard River	1984-1991	USACE-SAS & Soil Consultants	SYR-Series	33	SPT	32	24	-60 MLLW	No
Cooper River	1988	USACE-SAS	VC-5-#-88	2	UD	2	1	-66 MLLW	No
Ordnance Reach	1988	USACE-SAS	OR-#-88	11	SPT	9	6	-71 MLLW	No
Meyers Bend	1988-1990	USACE-SAS	MB-Series	3	SPT	3	3	-55 MLLW	No
Cooper River	1988-1990	USACE-SAS	CCR-Series	18	SPT	17	1	-62 MLLW	No
Filbin Creek	1988-1990	USACE-SAS & General Engineering	FCR-Series	16	SPT	9	5	-55 MLLW	No
Daniel Island	1988-1990	USACE-SAS	DIB-Series	9	SPT	9	6	-55 MLLW	No
Daniel Island	1988-1990	USACE-SAS	DIR-Series	13	SPT	13	11	-55 MLLW	No
Naval WPN Station	1988-1990	USACE-SAS	NYR-Series	9	SPT	8	6	-69 MLLW	No
Cooper River	1988-1990	USACE-SAS	PT-Series	12	SPT	12	12	-70 MLLW	No
Cooper River	1989-1990	USACE-SAS	NCR-Series	17	SPT	17	14	-55 MLLW	No
Shipyard River	1991	SM&E, Inc.	SYR-GOT-91	3	SPT	2	2	-79 MLLW	No
Cooper River	1994	Athena Technologies	CR-CH-94	11	Vibracore	4	0	-50 MLLW	No
Cooper River	1995	Athena Technologies	ECO-CH-95	18	Vibracore	N/A	N/A	TBD	No
Cooper River	1996	Athena Technologies	R-#-96	3	Vibracore	0	0	-38 MLLW	No
Naval WPN Station	1996-2009	USACE-SAC, General Engineering	NWS-Series	19	Vibracore	4	0	-47 MLLW	No
Cooper River	1997	General Engineering	PS-SI-#-97	2	Vibracore	0	0	-34 MLLW	No
Naval Complex	1997	General Engineering	MT-96 Series	8	Vibracore	0	0	-36 MLLW	No
Naval Complex	1997	General Engineering	DS-DD-97	8	Vibracore	0	0	-28 MLLW	No
Shipyard River	1998	USACE-SAS	SD-98-#	3	SPT	3	3	-80 MLLW	No
Shipyard River	1998	General Engineering	AT-#-98	2	Vibracore	0	0	-39 MLLW	No
Cooper River	1998	General Engineering	BM-S-#-98	2	Vibracore	0	0	-41 MLLW	No
Naval Complex	1999	General Engineering	NC-S-#-99	19	Vibracore	1	0	-46 MLLW	No
Shipyard River	1999	General Engineering	SC-S-#-99	3	Vibracore	N/A	N/A	NA	No
Cooper River	2000	General Engineering	CR-MS-#-00	3	Vibracore	0	0	-27 MLLW	No
Cooper River	2004	Athena Technologies	CR-DITB-04	4	Vibracore	2	0	-47 MLLW	No

[10] Common subsurface exploration methods used were Standard Penetration Test (SPT), Vibracoring, and Undisturbed (UD) Shelby Tube sampling

4.2.2. Lower Harbor Borings

There have been a total of 155 SPT and vibracore borings drilled in the lower harbor by USACE, Savannah Core Drill Unit, Athena Technologies, Inc., and General Engineering, Inc., from 1972 to 2004. The maximum penetration depth of these borings ranges from -43 feet to -71 feet MLLW. Of the 155 borings drilled, only 29 penetrate to the maximum proposed dredging depth of -52 feet MLLW (Table B-3). A review of the pertinent drilling logs indicates that no rock was encountered.

Table B-3. Catalogue of exploration drilling within the lower harbor reaches, Charleston Harbor.

Reach	Year	Driller	Boring ID	Total # Borings	Boring Type	#Drilled to -45 ft MLLW	#Drilled to -52 ft MLLW	Max depth drilled	Rock Sampled?
Wando River	1972-1998	USACE-SAS	WR-Series	25	SPT	18	6	-65 MLLW	No
Wando River	1979-1980	Soil Consultants	SCI-W-Series	21	SPT	5	4	-71 MLLW	No
Wando River	1981	USACE-SAS	WRB-#-81	20	SPT	20	2	-58 MLLW	No
Rebellion Reach	1988	USACE-SAS	RR-Series	8	SPT	8	1	-56 MLLW	No
Columbus TB	1988	USACE-SAS	CTB-#-88	12	SPT	10	0	-51 MLLW	No
Town Creek	1988-1994	USACE-SAS	MP-Series	3	SPT	3	1	-68 MLLW	No
Town Creek TB	1988-1994	USACE-SAS	CHR-Series	6	SPT	6	1	-57 MLLW	No
Horse Reach	1988-1994	USACE-SAS	HR-Series	6	SPT	6	1	-56 MLLW	No
Town Creek-Drum Island	1988-1994	USACE-SAS	TCU-Series	7	SPT	7	3	-69 MLLW	No
Town Creek	1988-1995	USACE-SAS	DI-Series	6	SPT	5	2	-58 MLLW	No
Hog Island	1988-	USACE-SAS	HI-Series	6	SPT	6	3	-67 MLLW	No
Town Creek	1988-1990	USACE-SAS	TWR-Series	13	SPT	13	1	-56 MLLW	No
Shutes Reach	1989-1994	USACE-SAS	SR-Series	3	SPT	3	1	-61 MLLW	No
Town Creek-Drum Island	1990	USACE-SAS	TCL-#-90	8	SPT	8	1	-56 MLLW	No
Daniel Island TB	1996	Athena Technologies	DI-TB-#-96	2	Vibracore	1	0	-45 MLLW	No
Town Creek Channel	1997	USACE-SAS	FR-#-97	2	SPT	1	1	-63 MLLW	No
Union Pier	2003	General Engineering	UPT-03	4	Vibracore	0	0	-43 MLLW	No
Town Creek TB	2004	Athena Technologies	CR-LTC-04	3	Vibracore	3	1	-56 MLLW	No

4.2.3. Upper & Lower Harbor Laboratory Soils Testing

There is very little existing geotechnical laboratory data that represents the present in-situ subsurface conditions within the navigation channels of Charleston Harbor. The material characterization discussed within this chapter relies heavily upon the visual classification documented in the historical boring logs (Attachment B-1). Historical test data that is available only represents sediment that has already been previously removed through dredging. Additional soils data should be collected during the Pre-Engineering and Design Phase of the project.

4.2.4. Upper & Lower Harbor Laboratory Rock Testing

There are no indications of bedrock units such as limestone, shale, or sandstone within any of the historical borings previously drilled within the upper and lower harbor. Likewise, there is no record of any rock testing having been conducted within either the upper or lower harbor. The only bedrock unit present in the project lies within the entrance channel, which is discussed in Chapter V.

4.2.5. Upper and Lower Harbor Geophysical Survey, 1994

Dr. Paul Gayes of Coastal Carolina University was contracted by USACE-Charleston District to conduct sub-bottom profiling in support of deepening the upper and lower harbor in 1994. The geophysical data were provided in hardcopy to the district, and were later scanned and imported into an ArcGIS format. Metadata indicate that shape files of the seismic lines and reflectors were created based upon timing, depth, location, and acoustic return assumptions; the actual values were not known or lost. Ambiguity also exists to depth accuracy of the reflectors observed in the 1994 profiles[11]. Therefore, only boring data is used to characterize subsurface conditions.

4.3 Analytical Methods

4.3.1. Historical Borings and gINT Database

A total of 549 drilling logs (Attachment B-1) were input into Bentley's gINT geotechnical software program, using a USACE report template. Boring elevations were corrected from MLW to MLLW using the conversion factor: 0.0 MLLW = -0.2 MLW. Borings without positive geographic control were not used. Furthermore, some of the drilling logs did not utilize the Unified Soils Classification System, but they were characterized and re-interpreted based upon USCS convention. For each SPT boring, the N-value[12] was calculated from blow-count information recorded on the original log.

4.3.2. Upper & Lower Harbor Subsurface Fence Diagram Development

Fence diagrams were created for each of the upper and lower harbor reaches using gINT geotechnical software. Historical borings (Attachment B-1) were input into the program in order to generate fence diagrams. Each fence diagram consists of a series of "stick logs" that show soil type, thickness, elevation and SPT N-value (for SPT borings only) within the subsurface. Generally, greater the coverage and density of boring data translates into a more accurate subsurface interpretation. The fence diagrams for each harbor segment are presented in Figures B-22 through B-39.

[11] Dr. Paul Gayes of CCU was contacted in January 2012, in order to provide technical advice regarding the 1994 dataset. He recommended that a newer geophysical survey should be run instead, given the inaccuracies involved with processing the 1994 dataset.

[12] The N-value is the sum of the blow-counts from the last 12-inches of penetration, out of an 18-inch drive. The blow-counts from the first 6-inches is normally discarded because the top 6-inches of the drive is considered to be the "seating drive" and the material sampled contains some loose fall-in material from the drilling. No correction factors are applied for SPT N values as they are a field measurement.

4.4. Upper Harbor Stratigraphy

4.4.1. Upper Harbor, Ordnance & Port Terminal Reaches

A total of 16 borings were used to describe the subsurface conditions within Ordnance and Port Terminal Reaches, shown in Figure B-22. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -56 feet MLLW. Significant amounts of shoaling are present along the eastern side of the turning basin. The maximum dredge depth within these channel segments is -54 feet MLLW which includes allowances for advanced maintenance and overdepth dredging (refer to Appendix A, Table 2.6.1). Within the proposed dredging prism, the soils are predominantly fine-grained and soft, with no evidence of limestone bedrock present. The material within the dredging prism consists of a very soft fat clay and elastic silt having variable thickness from station 1720+00 to station 1695+00. This stratum appears to overlie an interbedded sequence of very stiff lean silty clay and dense clayey sand from station 1690+00 to station 1665+00. This stiffer and denser material likely belongs to the Cooper Formation which extends into the proposed dredge prism. Borings indicate that no hard competent rock was encountered.

4.4.2. Upper Harbor, Filbin Creek Reach

A total of eight borings were used to describe the subsurface conditions within Filbin Creek Reach, shown in Figure B-23. Project surveys utilizing multi-beam sonar indicates that the present channel depth ranges in depth from -46 to -50 feet MLLW. The maximum dredge depth within this channel segment is -52 feet MLLW (refer to Appendix A, Table 2.6.1). Within the proposed dredging prism, the material is predominantly fine-grained and stiff, with no evidence of limestone bedrock present. The material within the dredging prism consists of stiff to very stiff lean silty clay that appears to grade laterally eastward into stiff elastic silt from station 1670+00 to station 1655+00. South of station 1655+00 this material appears to grade into an interbedded sequence of very stiff lean silty clay and dense clayey sand. This interbedded sequence of dense clayey sand and stiff lean silty clay extends from station 1655+00 to station 1620+00. Borings indicate that no hard competent rock was encountered.

4.4.3. Upper Harbor, North Charleston Reach

A total of 17 borings were used to describe the subsurface conditions within the North Charleston Reach, shown in Figure B-24. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -52 feet MLLW. The maximum dredge depth within this channel segment is -52 feet MLLW. Within the proposed dredging prism, the soils are predominantly fine-grained and stiff with no evidence of limestone bedrock present. The soils within the dredging prism consists of stiff lean silty clay and lean inorganic silt that is occasionally interbedded with dense clayey sand. A thick covering of soft to medium stiff elastic silt intermittently overlies the stiff silt and clay from station 1610+00 to station 1603+00 and from station 1575+00 to station 1570+00 in the eastern flank of the channel. The stiff lean silty clay and inorganic silt likely belong to the Cooper Formation which extends into the proposed dredge prism. Borings indicate that no hard competent rock in this channel segment.

4.4.4. Upper Harbor, Navy Yard Reach

A total of 14 borings were used to describe the subsurface conditions within the Navy Yard Reach, shown in Figure B-25. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -50 feet MLLW. The maximum dredge depth within this channel segment is -52 feet MLLW. The materials that lie within the proposed dredging prism are predominantly fine-grained and range from soft to stiff. Intermittent layers of granular material is generally medium dense. The material within the dredging prism consists of soft fat clay and silt, and stiff lean clay which is interbedded with medium dense clayey sand. The clayey sand is most prevalent between stations 1563+00 and 1540+00. Borings indicate that no hard competent rock was encountered.

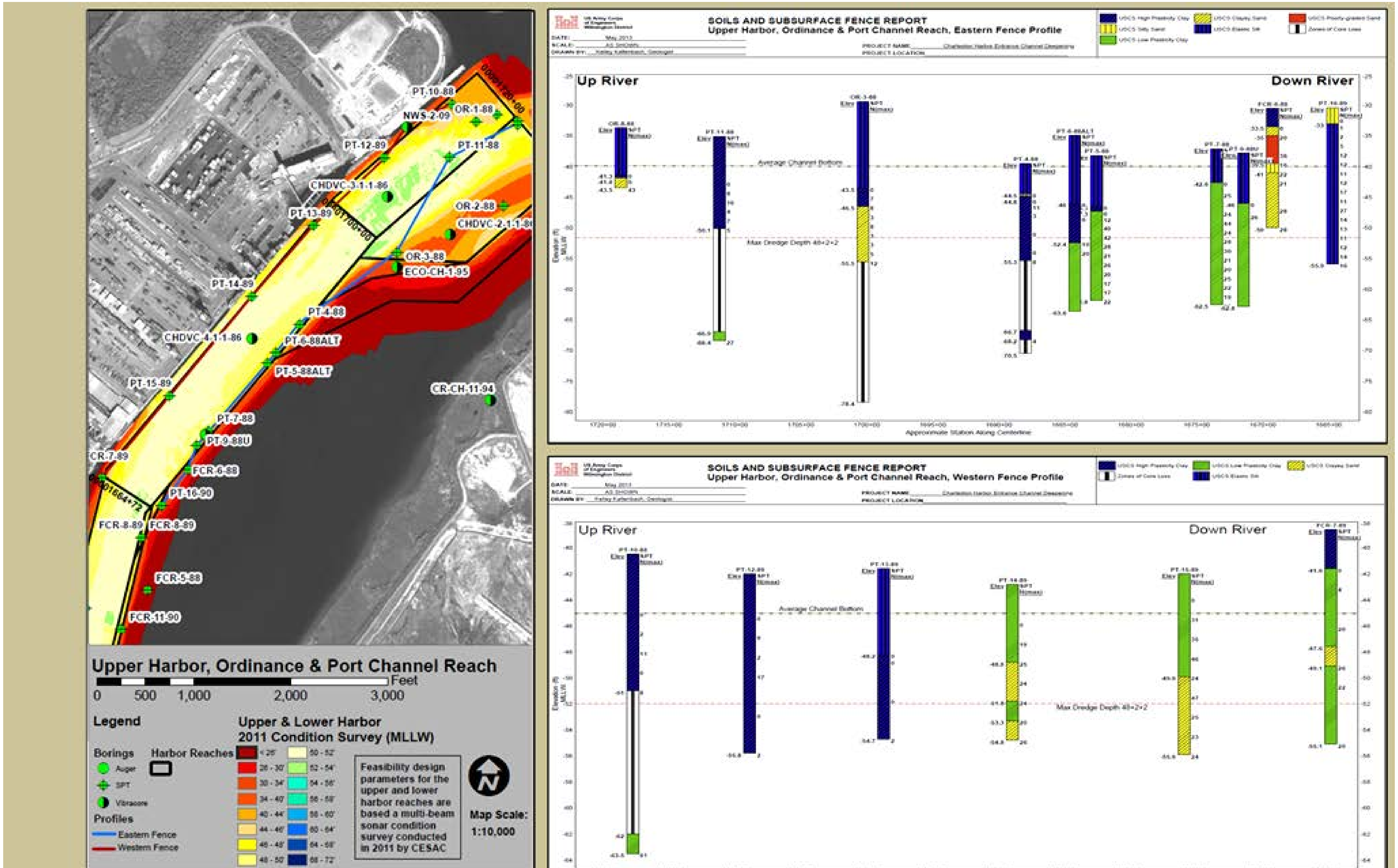


Figure B-22. Fence Diagram of Upper Harbor, Ordnance & Port Terminal Reach.

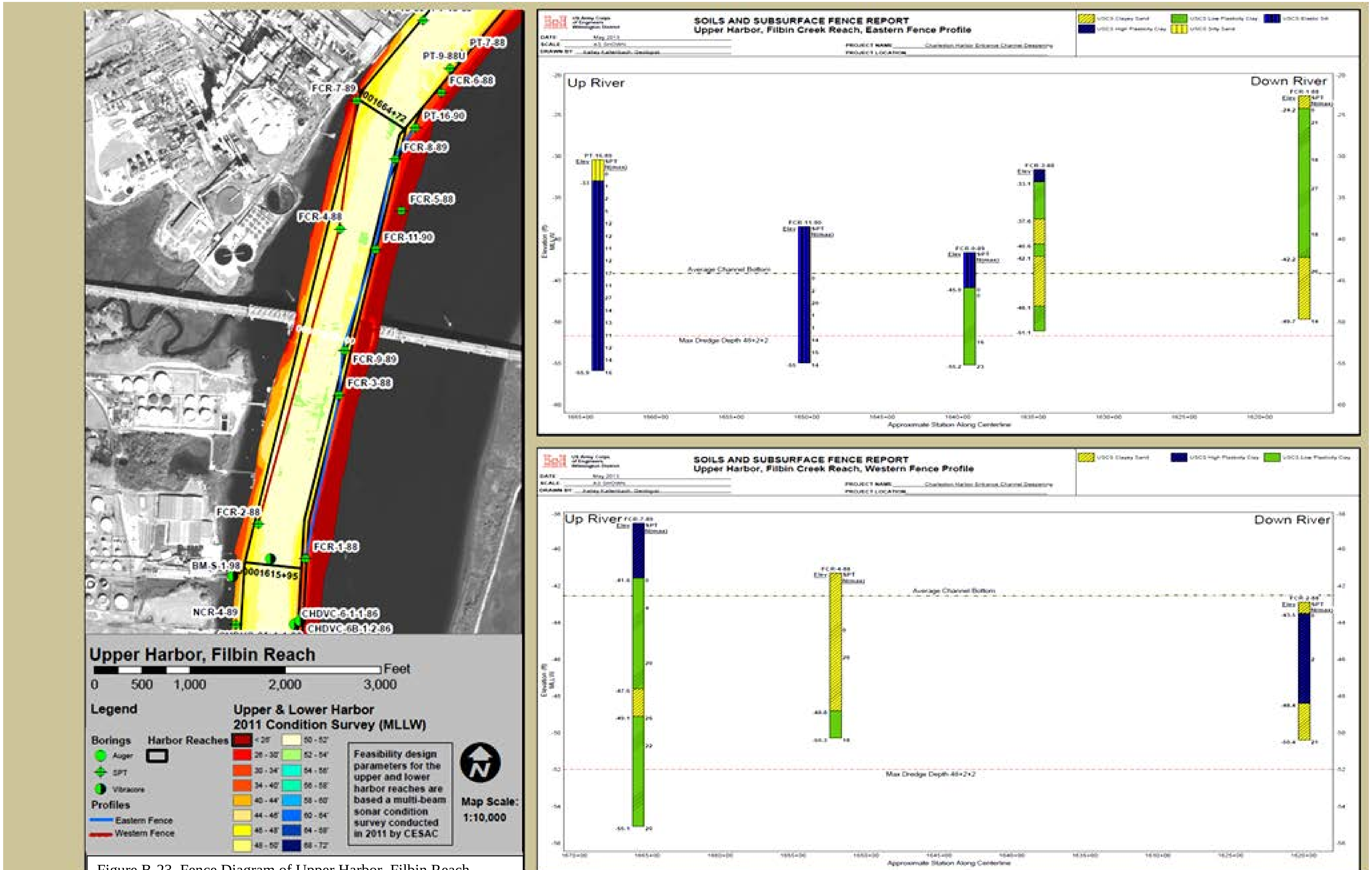


Figure B-23. Fence Diagram of Upper Harbor, Filbin Reach.

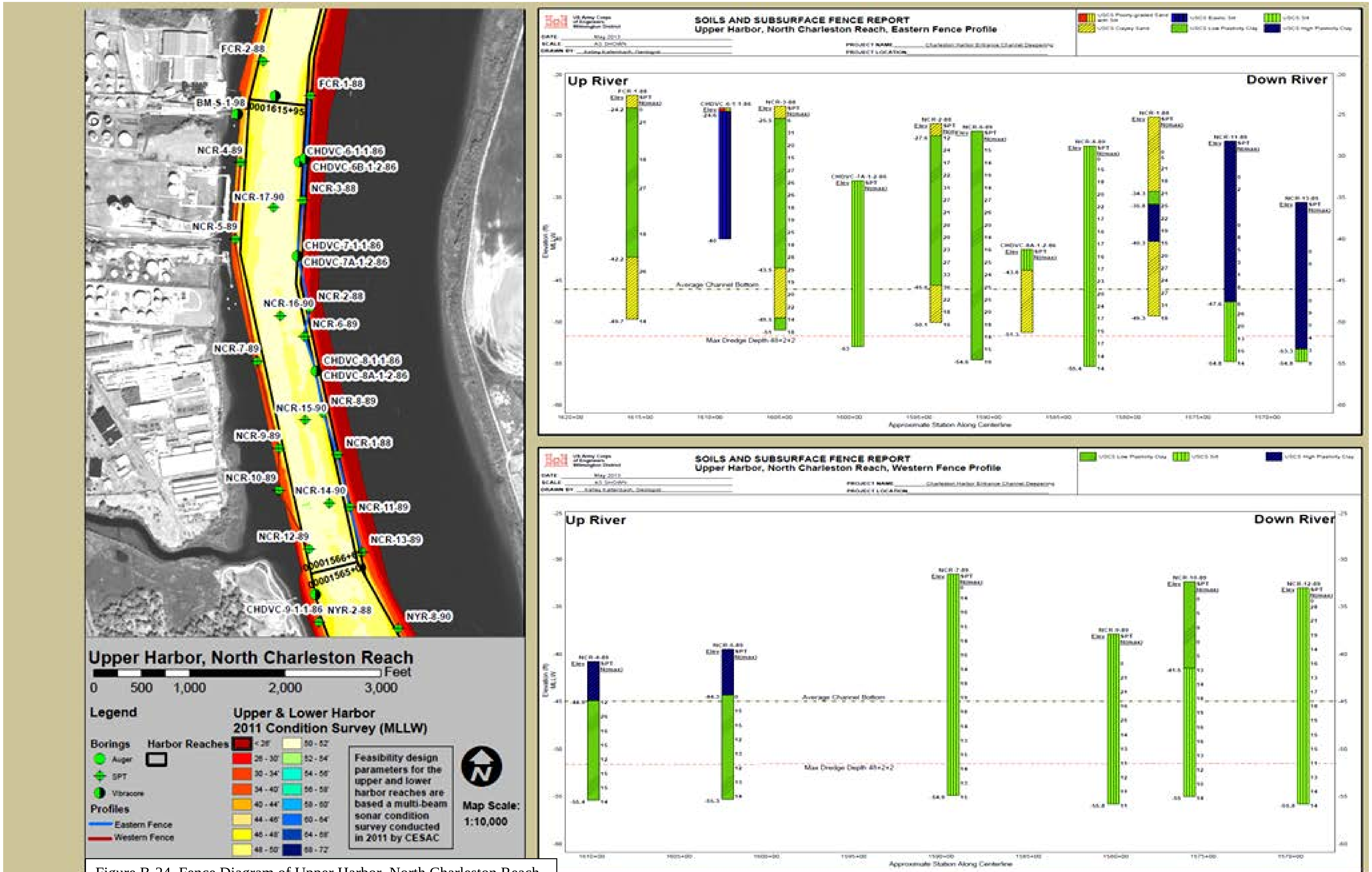


Figure B-24. Fence Diagram of Upper Harbor, North Charleston Reach.

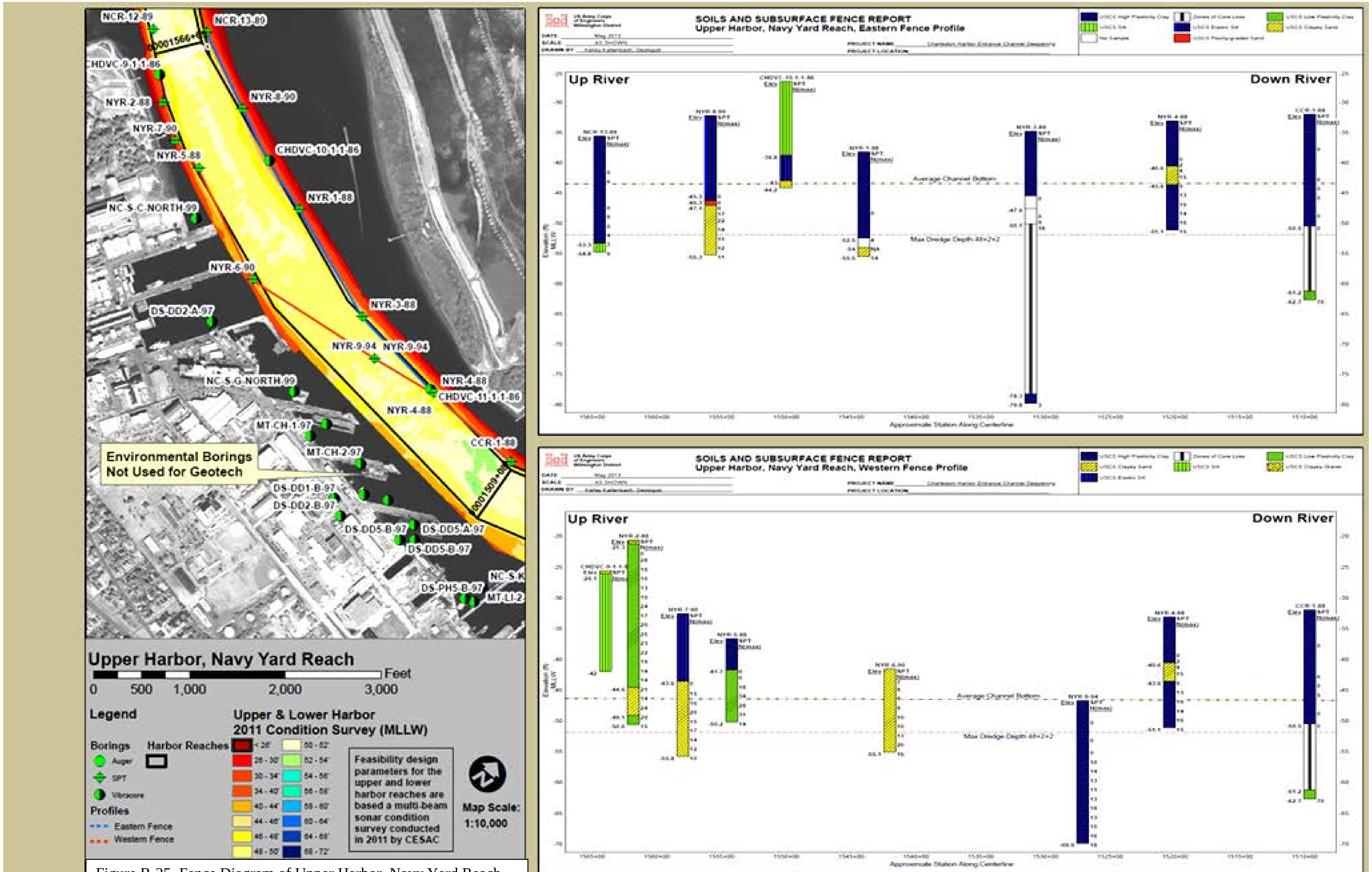


Figure B-25. Fence Diagram of Upper Harbor, Navy Yard Reach.

4.4.5. Upper Harbor, Clouter Creek Reach

A total of 17 borings were used to describe the subsurface conditions within the Clouter Creek Reach, shown in Figure B-26. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -54 feet MLLW. The maximum dredge depth within this channel segment is -52 feet MLLW. The material that lies within the proposed dredging prism is predominantly fine-grained and ranges from very soft to very stiff. Proceeding down the channel, these materials are comprised of very stiff inorganic silt, which grades laterally northward into soft fat clay from station 1500+00 to station 1485+00. A very dense cemented sand or possible outlier of bedrock is evident within the borings (CCR-17-90, CCR-3-88) from station 1490+00 to station 1487+00, on the south side of the channel. The materials between station 1485+00 and station 1465+00 consist of medium stiff fat clay occasionally interbedded with lenses of loose clayey sand. Southeast of station 1465+00 these materials contact a thick sequence of very stiff lean silty clay and inorganic silt, which interpreted to be a part of the Cooper Formation. Other than the cemented sands and very dense sandy soils encountered between stations 1490+00 and 147+00, there are no indications of hard competent rock present.

4.4.6. Upper Harbor, Daniel Island Bend & Reach

A total of 16 borings were used to describe the subsurface conditions within the Daniel Island Bend and Reach, shown in Figure B-27. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -54 to -40 feet MLLW, with the shallowest depths being south of station 1380+00. The maximum dredge depth changes from -52 feet to -56 feet MLLW south of station 1412+71. The material that lies within the proposed dredge prism is predominantly fine-grained and ranges from stiff to very stiff. Proceeding southward down the channel, the material consists of very stiff lean silty clay and lean inorganic silt and is interpreted represent the Cooper Formation exposed within the floor of the channel. These strata appear to be overlain by silty and clayey sands in the vicinity of stations 1390+00 and 1381+00. Borings indicate no hard competent rock is present in this channel segment.

4.4.7. Summary of Upper Harbor Stratigraphy within the Proposed Dredging Prism

The predominant soil types and SPT N-value range for each upper harbor reach are summarized in the table below.

Table B-4. Upper Harbor Stratigraphic Summary

Figure	**Reach**	**Predominant Soil**	**SPT-N (fine-grained)**	**SPT-N (granular)**
B-22	Ordnance & Port Terminal	Fat Clay, Lean Clay	0 - 40	3 - 47
B-23	Filbin Creek	Fat Clay, Lean Clay	0 - 22	0 - 26
B-24	North Charleston	Lean Clay, Inorganic Silt	6 - 26	18 - 22
B-25	Navy Yard	Fat Clay, Clayey Sand	0 - 15	10 - 22
B-26	Clouter Creek	Fat clay, Silt, Lean Clay	0 - 30	11 – 100
B-27	Daniel Island Bend & Reach	Inorganic Silt, Lean Clay	14 - 32	5 - 13

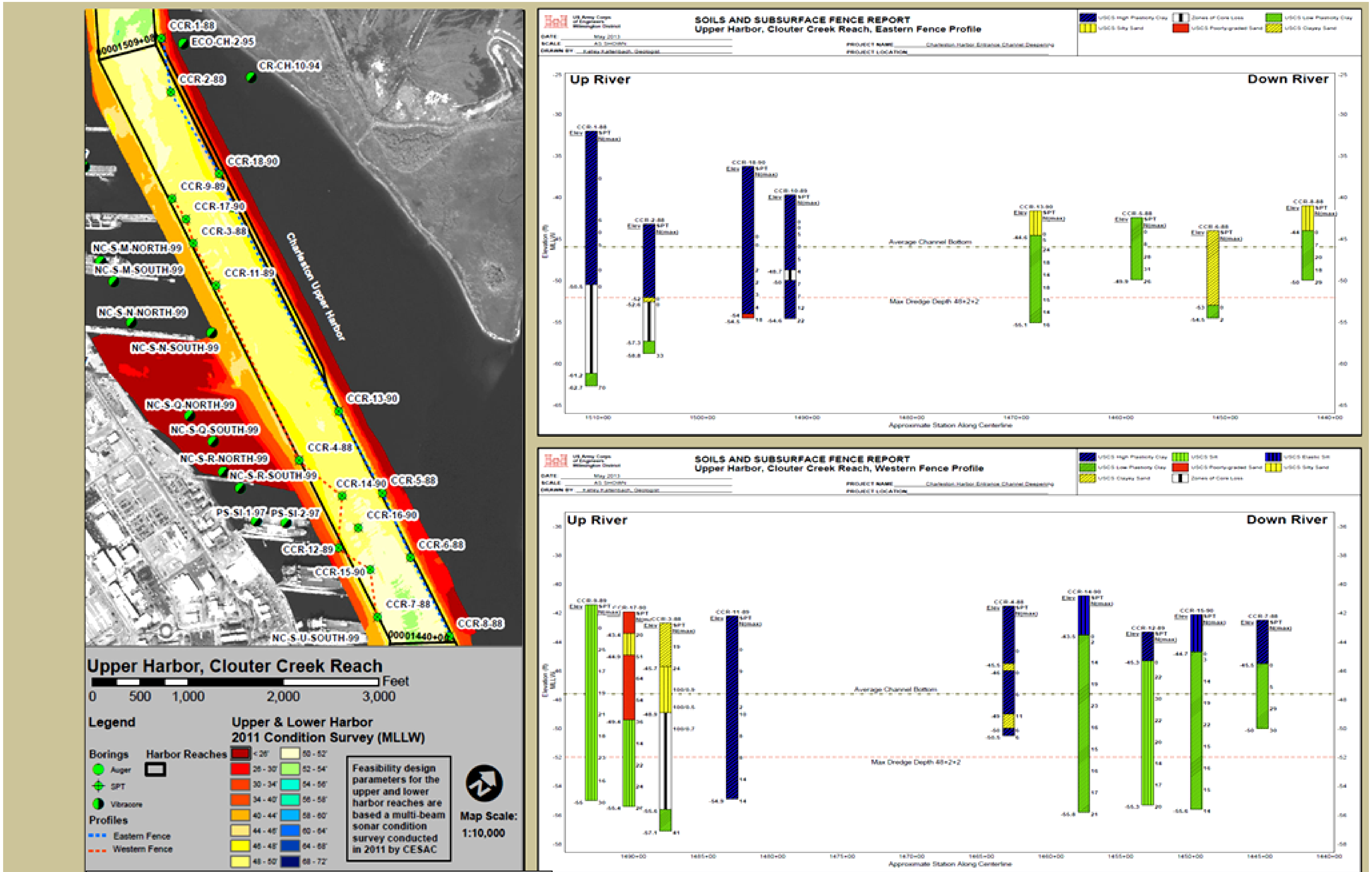


Figure B-26. Fence Diagram of Upper Harbor, Clouter Creek Reach.

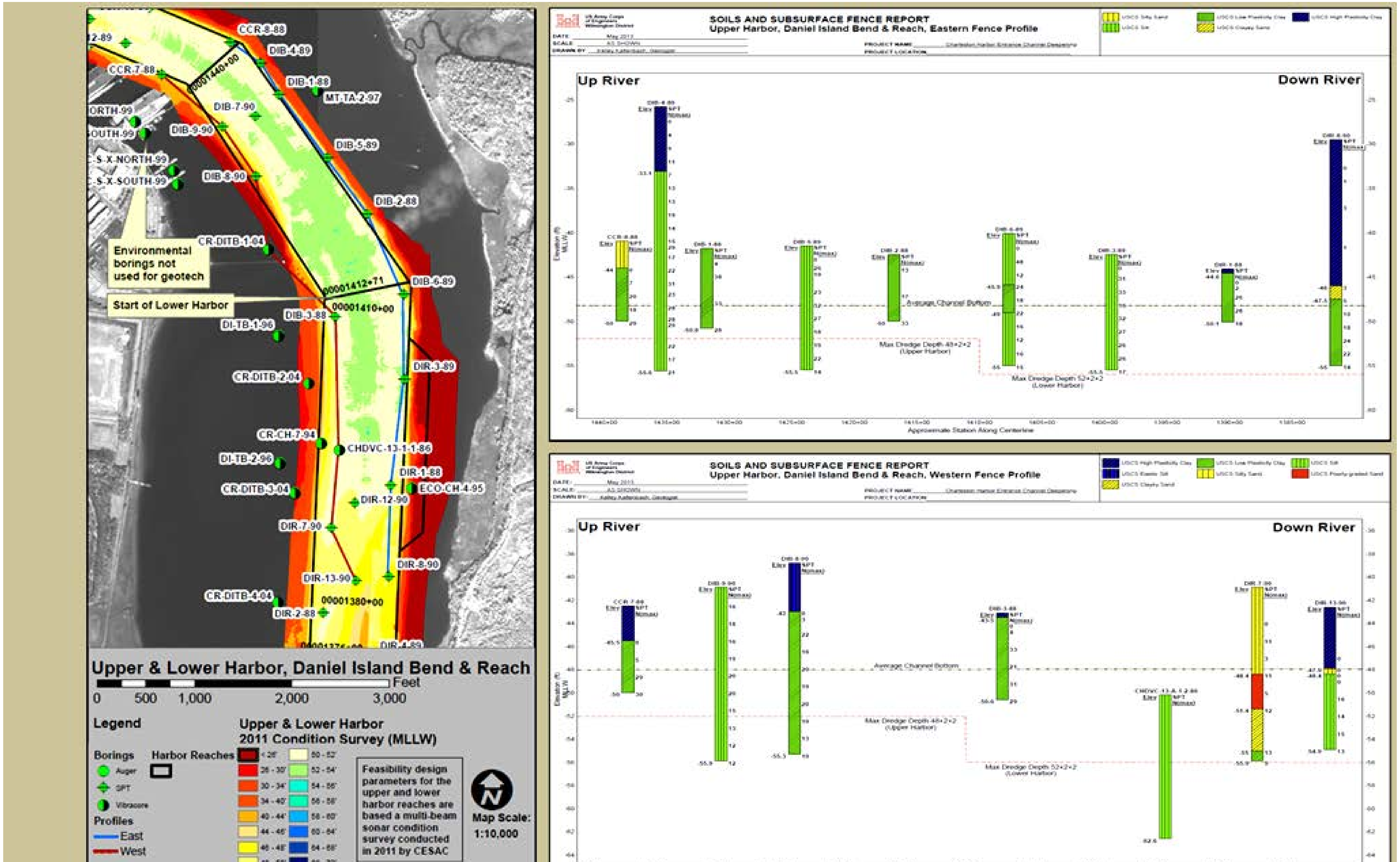


Figure B-27. Fence Diagram of Upper & Lower Harbor, Daniel Island Bend & Reach.

4.5 Lower Harbor Stratigraphy

4.5.1. Lower Harbor, Daniel Island Reach

A total of 14 borings were used to describe the subsurface conditions within the Daniel Island Reach, shown in Figure B-28. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -56 to -40 feet MLLW; the shallowest depths being south of station 1380+00 on the east side of the channel. The maximum dredge depth within this channel segment is -56 feet MLLW. The material that lies within the proposed dredge prism is predominantly fine-grained and ranges from stiff to very stiff. Proceeding down the channel, this material consists of very stiff inorganic silt and lean silty clay, interbedded with medium dense clayey sand from station 1400+00 to 1385+00. The clayey sand strata appears to pinch out south of station 1385+00, the soils becoming exclusively fine-grained and dominated by very stiff to hard inorganic silt and lean silty clay. The majority of the material within the dredge prism is likely part of the Cooper Formation. Borings indicate that there are no occurrences of hard competent rock in this channel segment, though the soils are quite stiff.

4.5.2. Lower Harbor, Myers Bend & Drum Island Reach

A total of 11 borings were used to describe the subsurface conditions within the Myers Bend and Drum Island Reaches, shown in Figure B-29. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -60 feet MLLW; the majority of both channels being deeper than -50 feet MLLW. The maximum dredge depth within this channel segment is -60 feet MLLW which includes a larger allowance for advanced maintenance and overdepth dredging (refer to Appendix A, Engineering, Table 2.6.1). Much of the new work material appears to lie along the sides of the channels. The center of both channels is generally within 6 feet of the -56 foot MLLW maximum dredging depth. The materials that lie within the channel segment are predominantly fine-grained and range in stiffness from very soft to hard. Proceeding down the channel, this material consists of very stiff to hard lean silty clay from station 1342+00 to 1325+00. These materials are very stiff and are interpreted to be part of the Cooper Formation, which extends into the proposed dredging prism. From station 1325+00 to station 1280+00, this stratum comes into contact with, and is overlain by, very soft, elastic silt and very loose clayey sand. Existing borings indicate that there is no hard competent rock in this channel segment.

4.5.3. Lower Harbor, Wando Upper Reach & Turning Basin

A total of 11 borings were used to describe the subsurface conditions within the Wando Upper Reach & Turning Basin, shown in Figure B-30. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -54 feet MLLW. Much of the channel and turning basin appear to be greater than -50 feet MLLW in depth. The maximum dredge depth within this channel segment is -58 feet MLLW which includes additional provision for advanced maintenance dredging. The materials that lie within the channel segment are predominantly fine-grained and range in stiffness from stiff to very stiff. Proceeding down the channel, these materials consist of stiff to very stiff lean silty clay from station 125+00 to station 105+00. These materials are interpreted to be part of the Cooper Formation, which extends into the proposed dredging prism. The stratum appears to pinch out or plunge into subsurface on the east side of the channel from station 105+00 to station 77+00; however along the western side, the strata is interbedded with medium dense clayey sand and very stiff fat clay from station 97+00 to station 85+00. Borings indicate no competent rock is present within this segment.

4.5.4. Lower Harbor, Wando Lower Reach

A total of 12 borings were used to describe the subsurface conditions within the Wando Lower Reach, shown in Figure B-31. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -52 feet MLLW. Much of the channel along centerline appears to be -48 to -50 feet MLLW in depth. The maximum dredge depth within this channel segment is -58 feet MLLW which includes additional provision for advanced maintenance dredging. The materials within the proposed dredging prism appear to be both coarse and fine-grained, with stiffness and density values ranging from very soft to very stiff and loose to medium dense, based upon SPT N-values. Proceeding down the channel, the material consists of stiff fat clay and elastic silt, which lies in contact with a sequence of interbedded silty-clayey sand near stations 3+50 and 12+00. The stratum grades laterally and down river from loose silty sand to medium dense clayey sand, then back into a loose silty sand from station 12+00 to station 35+00. From station 40+00 to station 65+00, the subsurface is predominantly composed of stiff elastic silt and soft organic clay. Existing borings indicate no competent rock is present in this channel segment.

4.5.5. Lower Harbor, Upper Hog Island Reach

A total of 10 borings were used to describe the subsurface conditions within the upper portion of Hog Island Reach, as shown in Figure B-32. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -56 feet MLLW. Much of the channel depth along centerline appears to be -46 to -50 feet MLLW, the deepest points (> -56 feet MLLW) located on the north end of the channel near station 1273+12. The maximum dredge depth within this channel segment is -56 feet MLLW. The materials that lie within the proposed dredging prism are both coarse and fine-grained, and have stiffness and density ranges from stiff to very stiff and loose to medium dense, based upon SPT N-values. Proceeding down the channel, this material consists of inorganic silt and elastic silt from station 1275+00 to station 1260+00. The materials pinch out or grade into an interbedded sequence of clayey and poorly graded sand that are present from station 1260+00 to station 1225+00. This sand stratum varies in density from loose to dense, based upon SPT N-values. From station 1225+00 southward the materials become finer grained. Existing borings indicate no competent rock is present in the channel.

4.5.6. Lower Harbor, Lower Hog Island & Horse Reaches

A total of 11 borings were used to describe the subsurface conditions within the lower portion of Hog Island & Horse Reaches, as shown in Figure B-33. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -60 feet MLLW. The deepest portion of the channel (-60 feet MLLW) lies between stations 1180+00 and 1175+00. The majority of the channel appears to have depths greater than -50 feet MLLW. The maximum dredge depth within this channel segment is -56 feet MLLW. Within the proposed dredging prism, the soils are predominantly fine-grained and soft. There is little information regarding the soils between -50 and -56 feet from station 1210+00 to station 1185+00, because the available

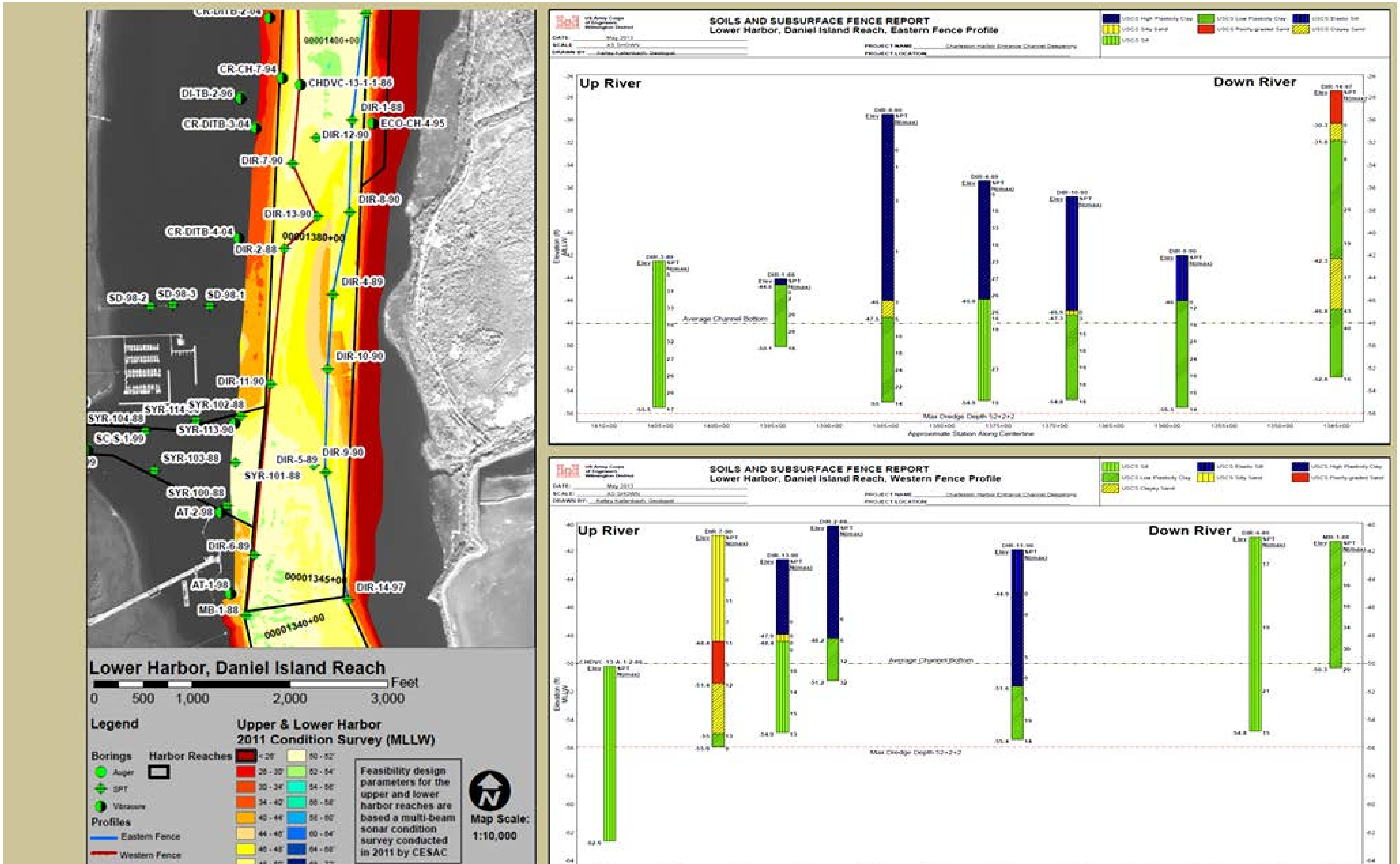


Figure B-28. Fence Diagram of Lower Harbor, Daniel Island Reach

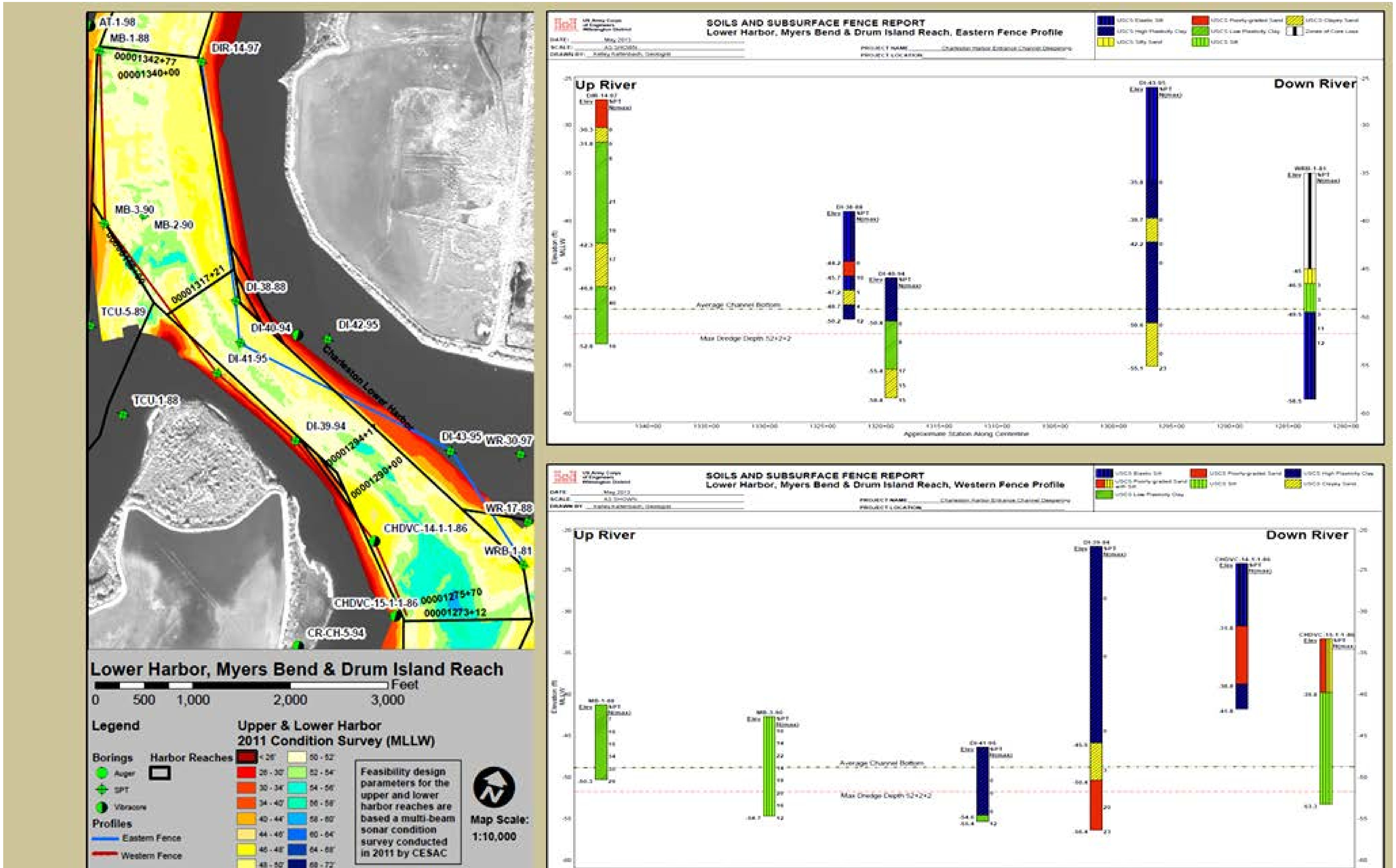


Figure B-29. Fence Diagram of Lower Harbor, Myers Bend & Drum Island Reaches

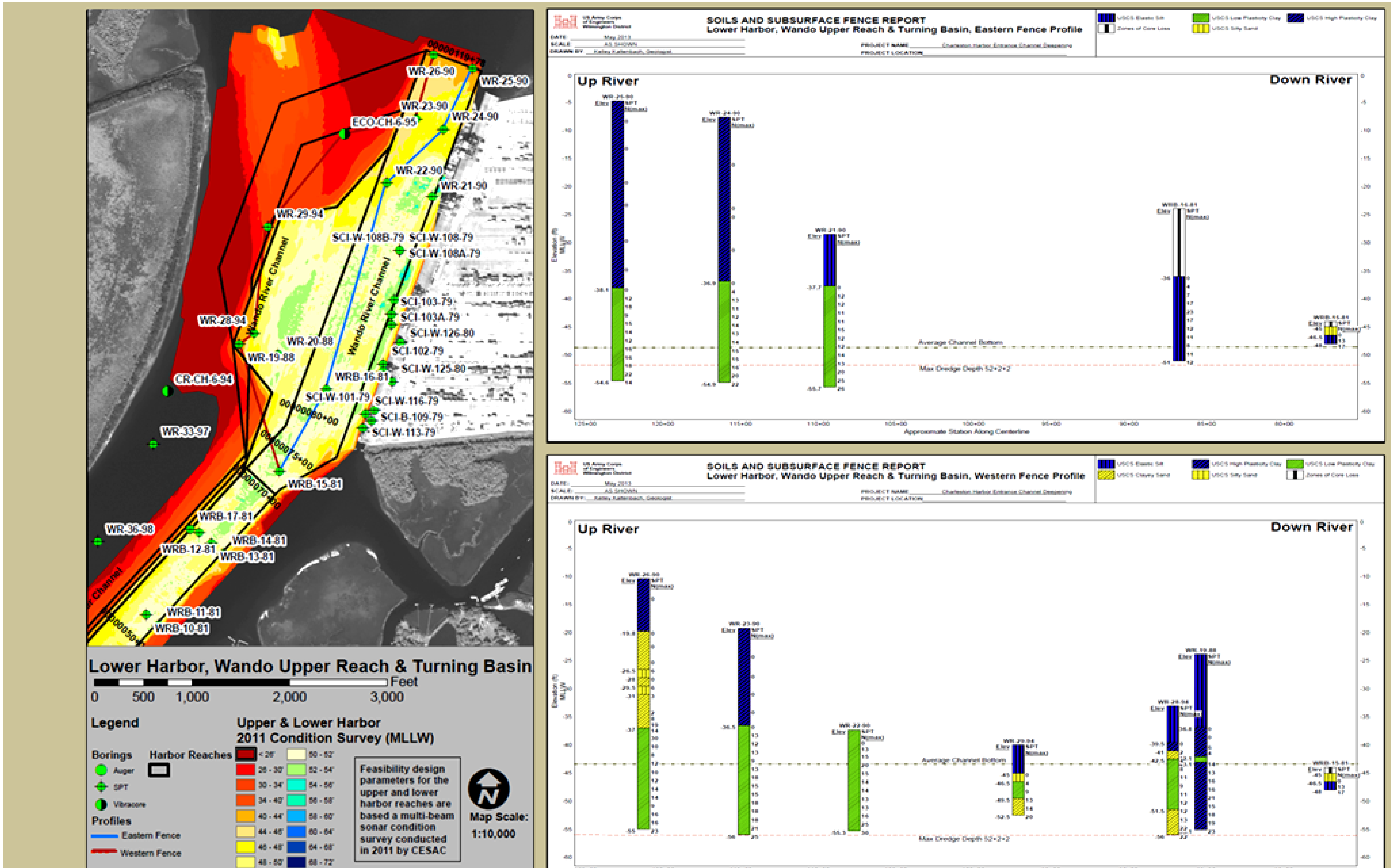


Figure B-30. Fence Diagram of Lower Harbor, Wando Upper Reach & Turning Basin

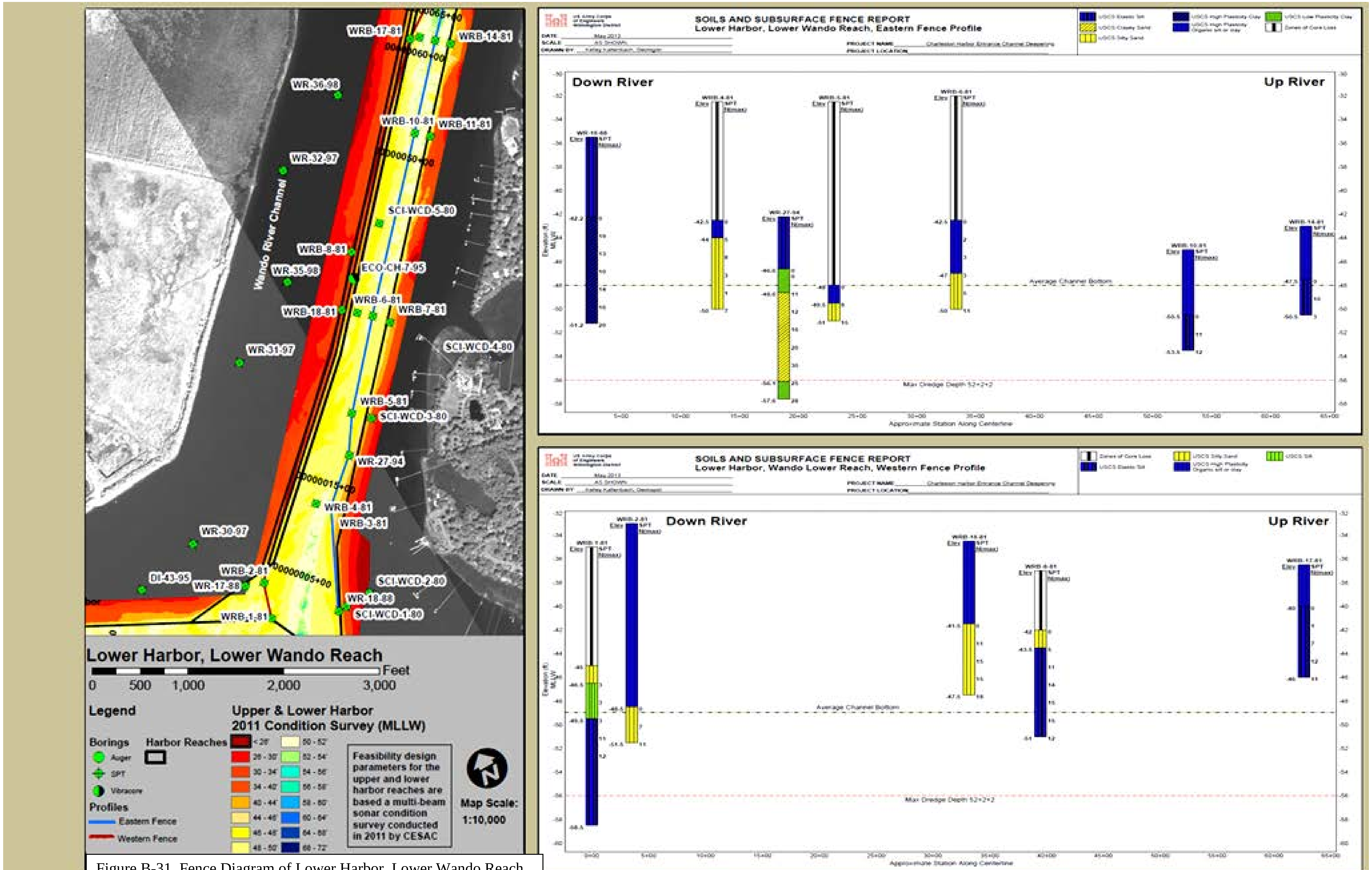


Figure B-31. Fence Diagram of Lower Harbor, Lower Wando Reach

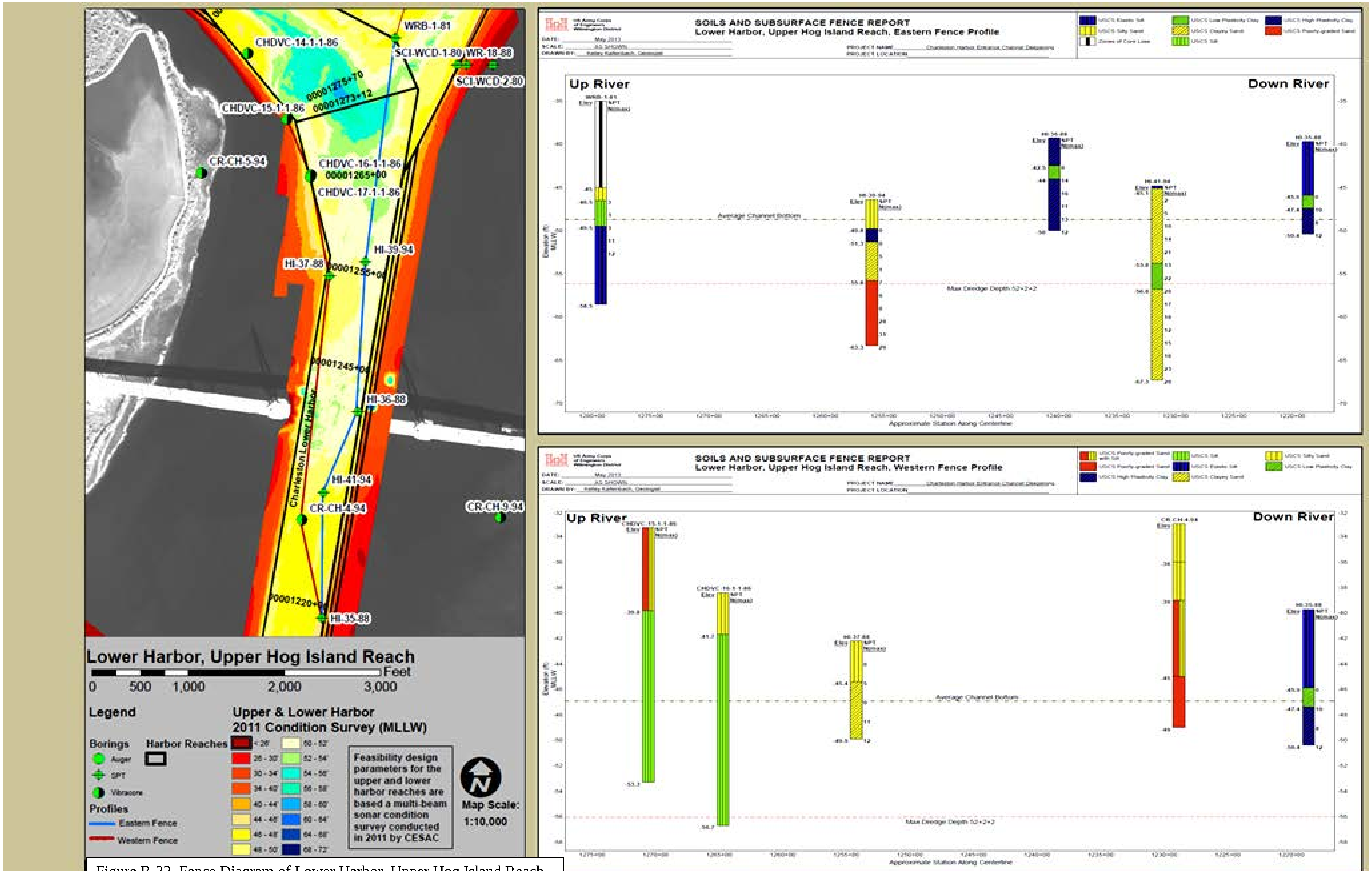


Figure B-32. Fence Diagram of Lower Harbor, Upper Hog Island Reach

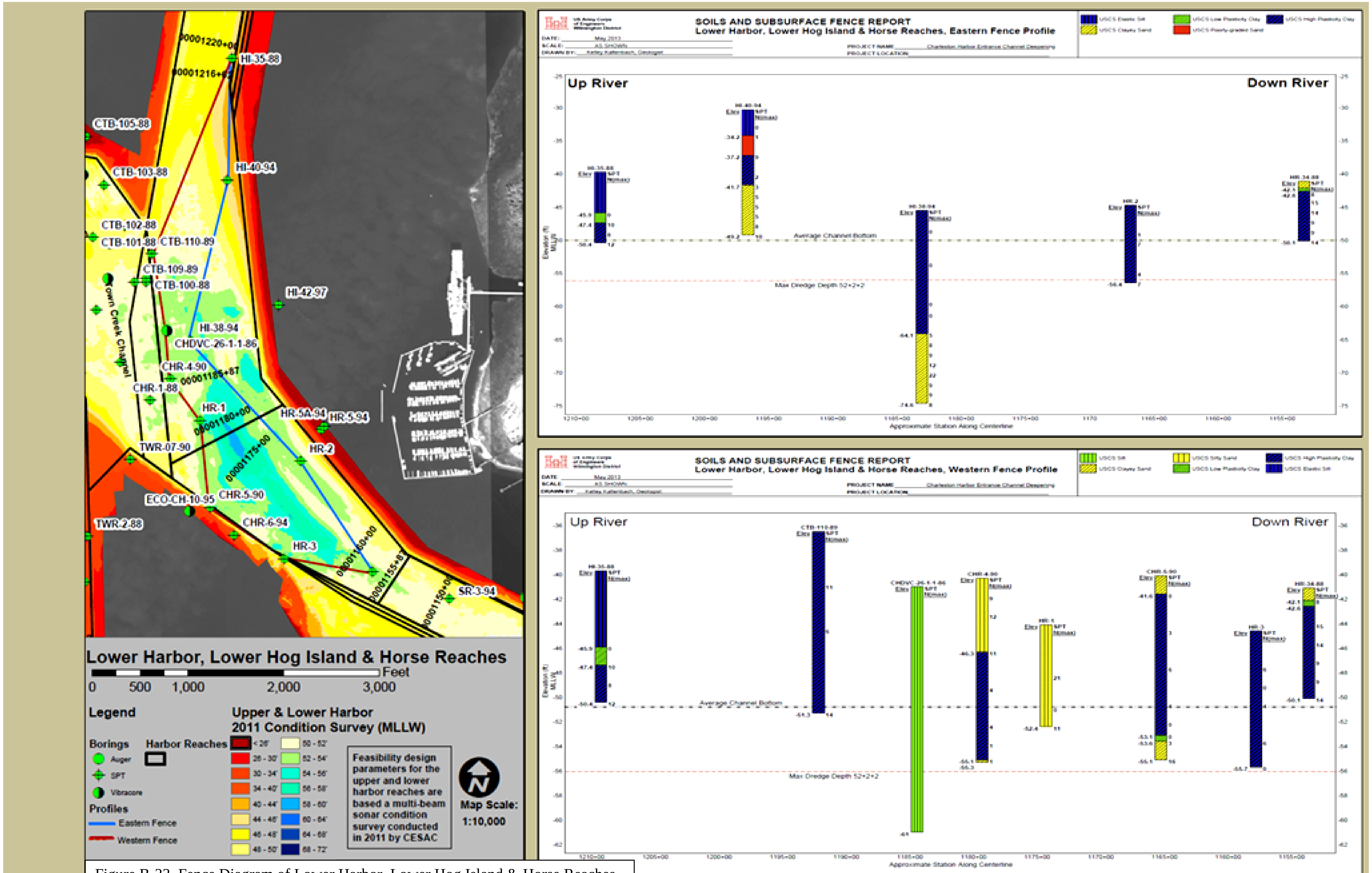


Figure B-33. Fence Diagram of Lower Harbor, Lower Hog Island & Horse Reaches borings do not penetrate to depth. However, based upon the boring data, the material is likely composed of soft silt and clay. Southeast of station 1185+00, the material consists of interbedded and mixed inorganic silt, fat clay, and clayey sands, which extends to station 1153+00. Existing borings indicate no competent rock is present in this channel segment.

4.5.7. Lower Harbor, Bennis Reach

A total of 8 borings were used to describe the subsurface conditions within the upper portion of Bennis Reach, as shown in Figure B-34. Project surveys utilizing multi-beam sonar indicates that the present channel ranges in depth from -46 to -50 feet MLLW, with a few areas showing erosional scour to -52 feet MLLW. The maximum proposed dredge depth within this channel segment is -56 feet MLLW. The proposed dredging prism is 6 to 10-feet thick containing materials that grade from fine to coarse-grained. Proceeding down the channel from station 1160+00, the material consists of intermittently stiff to soft fat clay and lean silty clay. The material grades laterally into a clayey to silty sand near station 1140+00. Between station 1140+00 and 1100+00 there is a lateral variation from silty sand to poorly-graded sand. Existing borings indicate no competent rock is present in this channel segment.

4.5.8. Lower Harbor, Rebellion Reach

A total of 15 borings were used to describe the subsurface conditions within Rebellion Reach as shown in Figure B-35. A composite fence diagram was drafted for the channel segment using borings from each side of the channel segment. Project surveys utilizing multi-beam sonar indicates that the present channel generally ranges in depth from -46 to -50 feet MLLW with a few areas along the channel centerline showing erosional scour to -56 feet MLLW. The maximum dredge depth within the Federal channel segment is -56 feet MLLW. The proposed channel dredging prism averages 7-feet thick. The material within the dredging prism ranges broadly from fine to coarse grained. From station 1070+00 to station 990+00, the material consists of inelastic silt, fat clay and lean clay, which is interbedded with poorly-graded sand and silty sand. SPT sampling indicates that the material is very stiff to hard between stations 1010+00 and 990+00. Strength values north of station 1020+00 are not well constrained due to lack of SPT N-values. The materials within the basin consists of a 4 to 5-foot thick bed of elastic silt and lean silty clay that overlies 8 to 16-foot feet of clayey to silty sand and poorly graded sand with silt, from station 1070+00 to station 1030+00. Based upon the existing drilling information, there is no hard or competent bedrock present within this channel segment

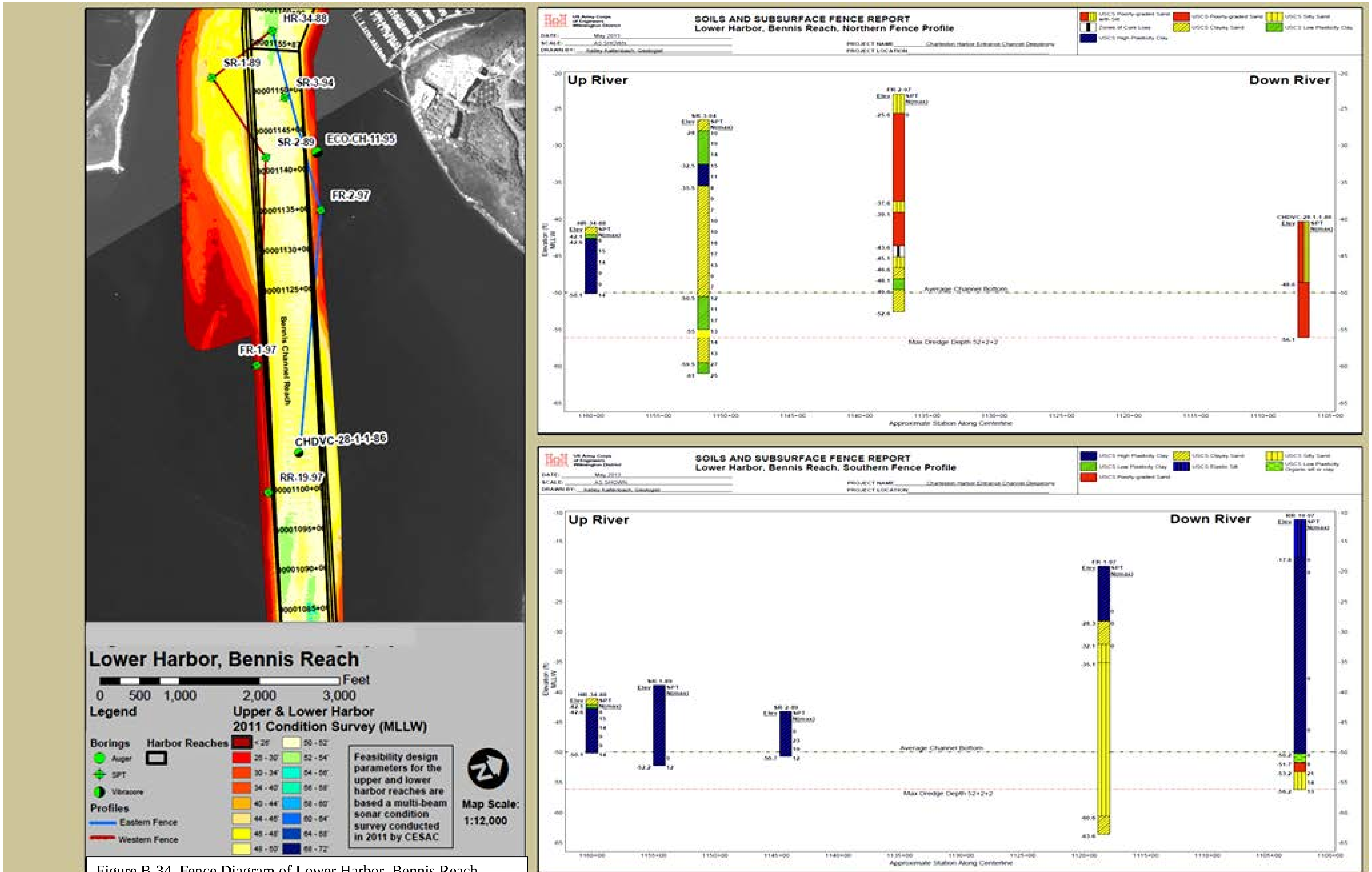


Figure B-34. Fence Diagram of Lower Harbor, Bennis Reach

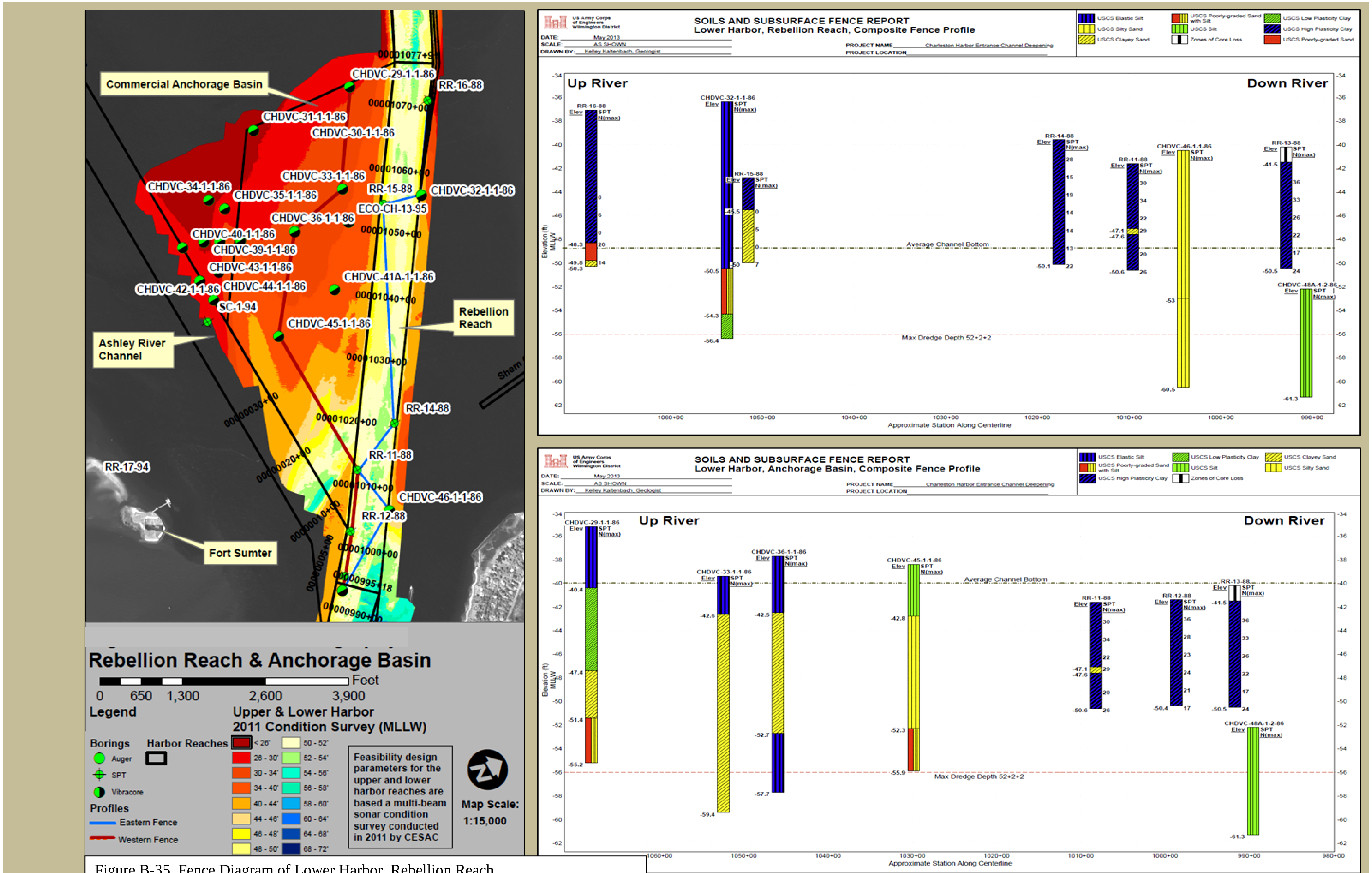


Figure B-35. Fence Diagram of Lower Harbor, Rebellion Reach

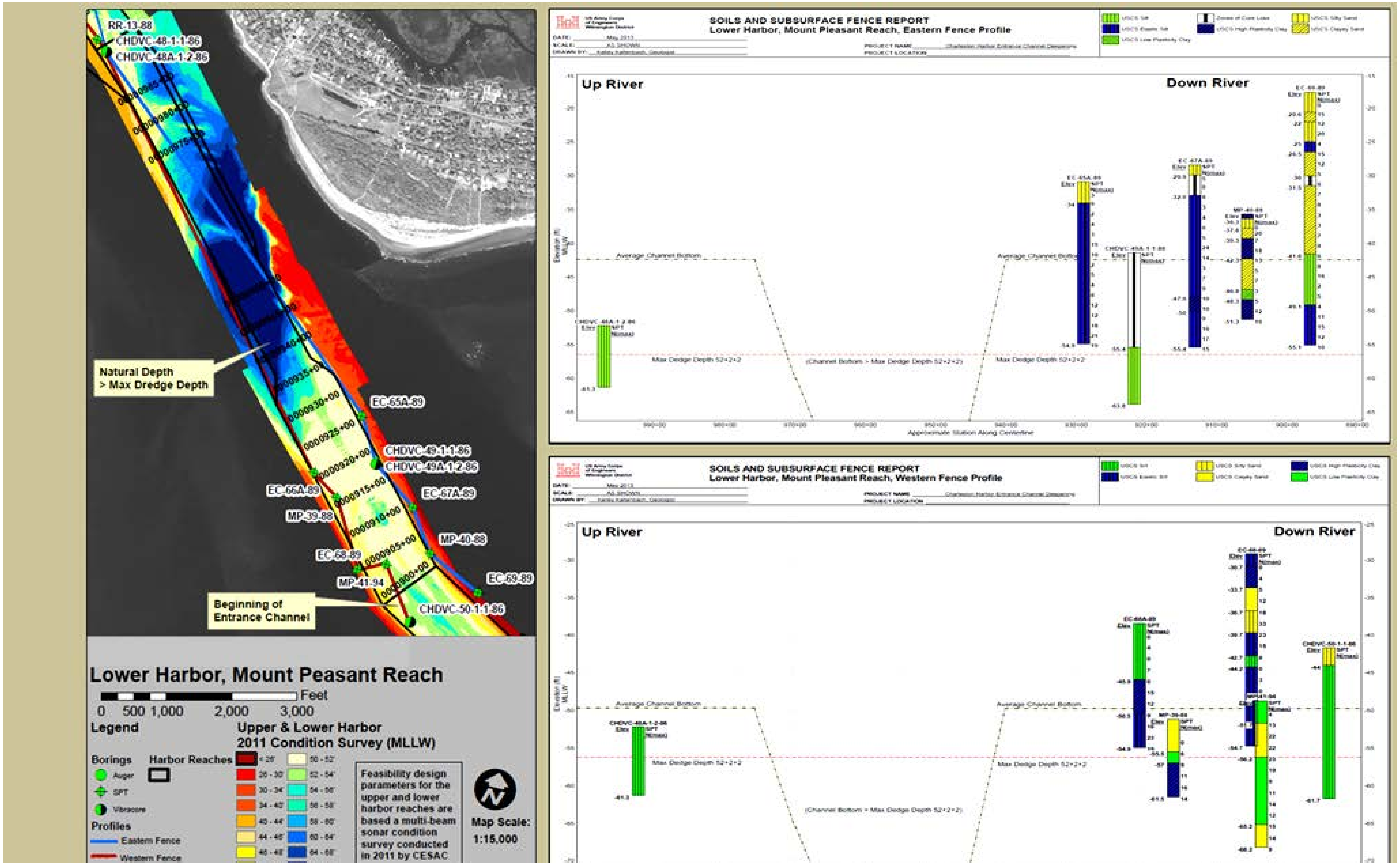


Figure B-36. Fence Diagram of Lower Harbor, Mount Pleasant Reach

4.5.9. Lower Harbor, Mount Pleasant Reach

A total of 11 borings were used to describe the subsurface conditions within Mount Pleasant Reach as shown in Figure B-36. Project surveys utilizing multi-beam sonar indicates that there is a significant amount of erosional scour between stations 975+00 and 935+00, in which the depths range from -56 to -72 feet MLLW. This area is naturally deeper than the proposed maximum dredge depth of -56 feet MLLW. South east of station 935+00, the channel bottom ranges in depth from -54 feet to -46 feet MLLW. The thickness of the material within dredging prism is approximately 12 feet. This material is generally fine-grained and is interbedded with lenses of granular material. North of station 980+00, the material is composed of inorganic silt. South of station 930+00 the material is composed of a laterally variable interbedded sequence of inorganic silt, clayey sand, clayey sand, and elastic silt that ranges in stiffness from very soft to very stiff. Existing borings indicate no competent rock is present in this channel segment.

4.5.10. Summary of Lower Harbor Stratigraphy within the Proposed Dredging Prism

The predominant soil types and SPT N-value range for each lower harbor reach are summarized in the table below.

Table B-5. Lower Harbor Stratigraphic Summary

Figure	Reach	Predominant Soil	SPT-N (fine-grained)	SPT-N (granular)
B-28	Daniel Island	Inorganic Silt, Lean Clay	5 - 26	12
B-29	Myers Bend	Lean Clay, Inorganic Silt	0 - 30	3
B-30	Wando River & Turning Basin	Lean & Fat Clay, Clayey Sand	9 - 25	0 - 20
B-31	Wando River	Fat Clay, Elastic Silt, Clayey Sand	3 - 12	1 - 11
B-32	Upper Hog Island	Inorganic Silt, Clayey Sand	3 - 12	1 - 21
B-33	Lower Hog Island & Horse	Fat Clay, Inorganic Silt, Silty Sand	1 - 6	3 - 16
B-34	Bennis	Fat & Lean Clay, Silty Sand	0 - 17	8 - 21
B-35	Rebellion	Clayey Sand, Fat & Lean Clay	17 – 36	0 - 14
B-36	Mount Pleasant	Lean Clay, Elastic Silt, Clayey Sand	9 - 23	0 - 22

V. SUBSURFACE INVESTIGATIONS ENTRANCE CHANNEL

5.1 General

The U.S. Army Corps of Engineers, Wilmington District, supported by Savannah District, conducted an extensive drilling and subsurface investigation within the Entrance Channel to Charleston Harbor, from August 10 to September 5, 2013 for Charleston District. A total of fifty borings were drilled within the existing channel, 2 to 14 miles offshore in water depths up to 60 feet, using USACE personnel and drilling equipment aboard Precon Marine's contracted jack-up vessel, *Cap'n Ray*. Borings were drilled to a maximum elevation of -63 feet MLLW in order to ascertain the physical characteristics of materials that lie within and below the proposed project dredging prism.

5.1.1. Purpose

The purpose of this chapter is to describe the efforts that were involved to locate, identify and determine the extent of rock within the entrance channel and summarize the results of the soil and rock testing. The results of the study are provided in order to refine the costs associated with deepening the harbor.

5.1.2. Scope.

The scope of the 2013 exploratory drilling investigation included the following;

- The drilling of a maximum number of 55 borings to log the subsurface stratum, collect SPT blow data, and recover intact rock cores for logging and lab testing.
- Submit representative rock samples to a USACE-approved geotechnical lab for unconfined compressive and splitting tensile strength (Brazilian method) testing.
- Submit representative unconsolidated material samples to a USACE-approved geotechnical lab for gradation and visual classification.
- Develop drilling logs, maps and cross-sections to characterize the investigated subsurface conditions within the entrance channel;
- Conduct an engineering analysis of the laboratory and field test results, and make recommendations to the PDT as to the best method of rock removal for the proposed deepening project.
- Provide engineering input to better refine material excavation quantities, excavation method, and ultimately, the feasibility cost estimate for construction.

5.1.3. Location of the Entrance Channel.

The Charleston Harbor Entrance Channel is located 1 to 14 miles offshore from the mouth of the harbor (Figure B-37). For the exclusive purposes of the geotechnical subsurface characterization, the channel was sub-divided into 1-mile long segments designated EC-1 through EC-21[13]. The 2013 subsurface investigation was conducted only within the entrance channel, specifically in the areas designated red. These areas were identified prior to drilling as having bedrock within the proposed dredge prism.

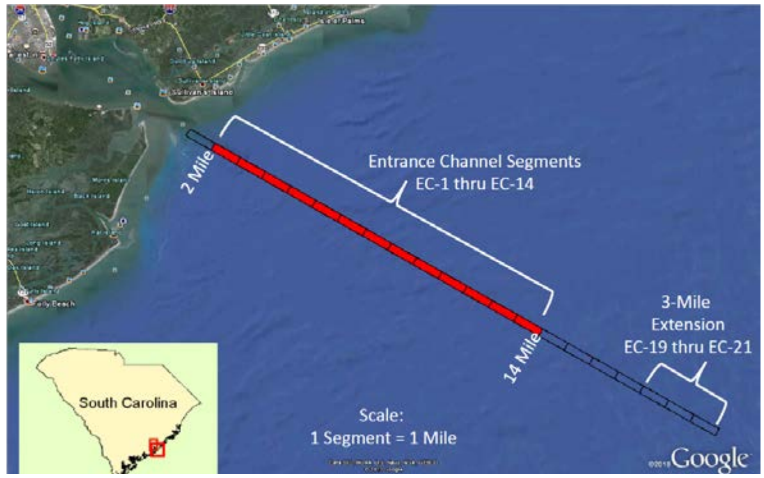


Figure B-37. Location of entrance channel, areas drilled in 2013 are colored red.

5.1.4. Entrance Channel Existing Conditions.

The entrance channel, Fort Sumter Reach has an authorized depth of -47 feet (MLLW) and extends from the 47-foot ocean contour through the jetties. The existing Federal channel is 1000 feet wide and is designed to have 4H: 1V side slopes. The mean tidal range, reported from Shipyard River, is 5.3 feet above mean low water, while the spring tide is 6.1 feet above mean low water. Bathymetric surveys (2011-2013) indicate that the entrance channel presently ranges in depth from 48 to 56 feet MLLW. Outside of this channel the surrounding seafloor deepens from -7 feet nearshore to -54 feet MLLW 17 miles offshore at the mouth of the channel. Condition surveys from 2011 and 2013 indicate that there are a series of small-scale bathymetric features located within the navigation channel between segments EC-17 and EC-21 (Figure B-38, Plate 1). Little shoaling was evident between the two condition surveys, which suggests that there is little active sedimentation within outer segments of the channel.

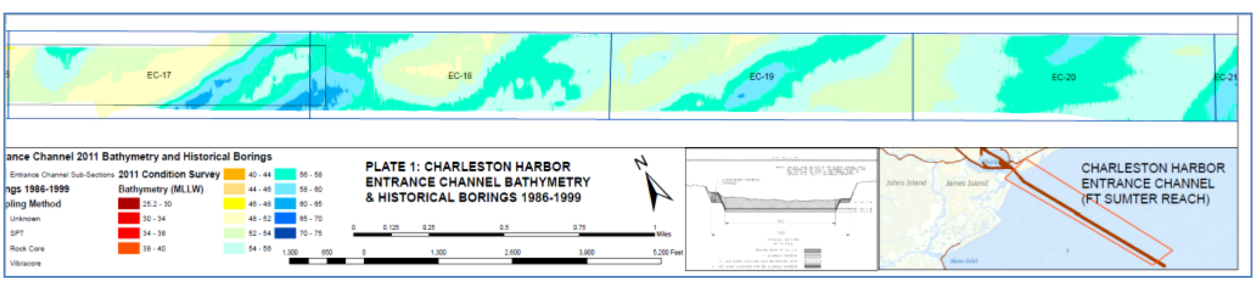


Figure B-38. Entrance channel bathymetric features located in the 3-mile extension.

[13] The designation of EC-1 through EC-21 is specific to the geotechnical investigation and does not apply to the remainder of the feasibility document. Subdividing the entrance channel was deemed necessary by the geotechnical team in order to efficiently characterize subbottom conditions and provide relatively quick reference points.

5.1.5. Unknowns

Prior to the 2013 investigation, there were only 193 borings drilled in the 17-mile long Charleston Harbor Entrance Channel. These drilling logs are provided in Attachment B-2. The boring density within this portion of the project was 193 borings/2070 acres, or 0.09 boring/acre of ocean floor. This did not include the proposed 3-mile extension. Furthermore, it was found that of the 193 borings historically drilled; only 22 penetrate into the new work dredge prism. Lastly, there were only six unconfined compressive strength tests conducted by USACE on record for the project. The subsurface investigation that was undertaken in August 2013 attempts to address the following unknowns:

- Location of significant amounts of rock;
- Type, characteristics and strength of rock;
- Depth constraints of the bedrock;
- Better define the area(s) in which the bedrock occurs.

5.2 Previous Supporting Investigations

5.2.1. 1986 OSI Exploration.

A total of 95 vibracores were drilled by Ocean Surveys Incorporated (OSI) within the Charleston Harbor Entrance Channel in 1986 (Attachment B-2). The purpose of the investigation was to determine the subsurface conditions in order to evaluate the feasibility of deepening the channel to -44 feet MLLW. The Cooper Formation was encountered within all the vibracores, and it was generally described as a consolidated, fine-grained, impure calcareous, glauconitic deposit having phosphate nodules. The material was described as olive-brown, clayey silt (MH/ML) with occasional layers of very silty, clayey fine sand (SM/SC). The unconfined compressive strength of the material was estimated to be 2-3 tons/square feet, based upon other engineering projects within the area. OSI estimated that the Cooper Formation was approximately 200 feet thick, and had experienced pre-consolidation pressures averaging 6 tons/square foot.

OSI also encountered limestone, which they termed “coquina”. The “coquina” was described as a light gray calcareous cemented sandy shell hash, which overlies the Cooper Formation in borings CHDVC-55 thru 57, 59, 60, 89, and 62. The unit was reportedly encountered at a depth of -32 MLW (-32.2 MLLW) in boring CHDVC-55, then dips southward to -45 MLW (-45.2 MLLW) in boring CDHVC-66. The material was found to be extremely hard due to cementation and well worked from wave action. OSI reported that the coquina would be the most difficult material to dredge, and this material would be encountered from the jetties seaward to the mouth of the entrance channel. The coquina consists of zones of very hard material interbedded with looser material, which was considered to pose a challenge to commercial dredging capabilities at the time.

Table B-6. Summary of historical subsurface investigations conducted within the entrance channel.

Harbor Channel	Year Drilled	Agency	Number Borings	Type Borings	Max Depth	Proposed Project Depth	# Boring Advanced to Depth	Rock Sample?	Rock Strength Test?
Entrance	1986	OSI[14]	95	Vibracore	-64 MLLW	-58 MLLW	32	Yes	No
Entrance	1988	SAS[15]	40	SPT	-52 MLLW	-58 MLLW	0	Yes	No
Entrance	1989	SAS	18	SPT	-56 MLLW	-58 MLLW	0	No	No
Entrance	1990	SAS	78	SPT	-55 MLLW	-58 MLLW	0	Yes	Yes
Entrance	1997	SAS	13	SPT	-62 MLLW	-58 MLLW	2	No	No
Entrance	1998	SAS	6	SPT & RC[16]	-61 MLLW	-58 MLLW	2	Yes	No
Entrance	1999	SAS	4	SPT & RC	-65 MLLW	-58 MLLW	1	Yes	No

5.2.2 USACE, SAS Drilling Program 1988-1999.

The U.S. Army Corps of Engineers, Savannah Core Drill Unit, drilled 159 borings within the Charleston Harbor Entrance Channel from 1988 to 1999 (Attachment B-2). The borings were drilled by the SPT Method (ASTM D-1586-11) using continuous sampling depth intervals of 1.5 feet to recover material samples and determine their strength properties. When rock was encountered, the driller switched over to rock coring methods to pull lengths of rock core for study. Historical review of the boring data indicates that although rock was sampled in a handful of the borings, very few (5) borings drilled penetrated to the presently proposed new work depths of -56 MLLW or -58 MLLW (see Table B-6 and Table B-7).

Table B-7. USACE rock sampling and testing in the Charleston Harbor Entrance Channel.

Boring #	Depth to Rock (MLLW)	Terminated Depth (MLLW)	Sample #	Sampled Depth (MLLW)	UCS (psi)	Rock Type	Strata Thickness
EC-59A-90	-45.8	-49.6’	1	47.3-47.7	114	Limestone	Unknown
EC-78A-90	-32.7	-50.2’	1	35.3-35.5	40	Limestone	6.4’
EC-134A-90	-36.3	-50.1	1	36.5-37.0	62	Limestone	4.3’
EC-140-90	-40.4	-50.5	1	41.9-42.5	120	Limestone	Unknown
EC-140-90	-40.4	-50.5	2	43.6-44.5	131	Limestone	Unknown
EC-138-90	-42.8	-49.4	1	44.4-44.8	69	Limestone	4.5’
EC-57A-90	-47.6	-50.0	2	47.6-50.0	---	Limestone	2.4’
EC-21-88	-48.9	-49.9	---	---	---	Limestone	Unknown
EC-22-88	-48.3	-50.0	---	---	---	Limestone	Unknown
EC-23-88	-44.0	-50.0	---	---	---	Limestone	Unknown
EC-24-88	-45.1	-51.1	---	---	---	Limestone	Unknown
EC-24-88A	-52.8	-56.6	---	---	---	Limestone	Unknown
EC-27-88	-49.6	-51.1	---	---	---	Limestone	Unknown
EC-28-88	-47.4	-49.7	---	---	---	Limestone	Unknown
EC-29-88	-43.7	-49.7	---	---	---	Limestone	Unknown
EC-29-88A	-44.0	-51.3	---	---	---	Limestone	Unknown
EC-30-88	-48.4	-50.8	---	---	---	Limestone	Unknown
EC-31-88	-47.8	-50.8	---	---	---	Limestone	Unknown
EC-33-88	-50.0	-50.5	---	---	---	Limestone	Unknown

[14] Ocean Surveys Incorporated (OSI)
[15] Savannah District, USACE (SAS)
[16] Rock Core (RC)

Boring #	Depth to Rock (MLLW)	Terminated Depth (MLLW)	Sample #	Sampled Depth (MLLW)	UCS (psi)	Rock Type	Strata Thickness
EC-57-88	-41.2	-50.2	---	---	---	Limestone	Unknown
EC-61-88	-45.3	-49.8	---	---	---	Limestone	Unknown
EC-63-88	-48.1	-50.6	---	---	---	Limestone	Unknown
EC-86-89	-50.0	-54.9	---	---	---	Limestone	Unknown
EC-87-89	-48.3	-54.8	---	---	---	Limestone	Unknown
EC-87A-90	-47.8	-50.2	---	---	---	Limestone	Unknown
EC-55-88	-48.3	-49.8	---	---	---	Limestone	Unknown
EC-112A-90	-47.9	-49.2	---	---	---	Limestone	Unknown
EC-139-90	-46.8	-49.6	---	---	---	Limestone	Unknown
EC-154-98	-51.6	-62.2	---	---	---	Limestone	8.7’
EC-158A-99	-45.7	-54.7	---	---	---	Limestone	Unknown

Limestone bedrock was encountered in 28 borings between depths -32.7 and -52.8 MLLW (Table B-7). The stratigraphic boundaries and thickness of the limestone (Edisto Formation) is not well constrained by the borings. Only borings EC-78A-90, EC134A-90, and EC-138-90 intersect what may be the lower contact between the limestone and the finer grained material of the Cooper Formation. These borings may have only sampled the thinnest lateral extent of the limestone strata. Had the remaining borings been advanced to depths that intersect the underlying Cooper Formation, the thickness of the overlying Edisto Formation might be known.

A total of six, 4-inch rock core samples were submitted to the SAD Geotechnical Testing Laboratory for petrographic analysis and unconfined compressive strength testing (UCS). The bedrock was sampled from stratum ranging from -35.1 to -47.7 MLLW. The laboratory verified the rock as limestone, consisting of 47-82% calcite, with the remaining material comprised of insoluble material. The lab described the limestone as being very light to medium gray in color, crumbly to soft to moderately hard, sandy, fossiliferous, and porous. Unconfined compressive strength tests ranged from 40 psi to 131 psi, indicating soft to very soft bedrock.

5.2.3. NOAA Diver Survey of Hardbottom Habitat, 1998

The National Oceanic and Atmospheric Administration identified four rock pinnacles within entrance channel segments EC-2 and EC-3 (Plate 5) during a diver survey of hardbottom habitat in August, 1998. The rock pinnacles were described as being comprised of “porous rock” and were ridge-shaped. Boring EC-158-98, which is located closed to pinnacle 4 contains sandy to shelly limestone. The rock pinnacles are interpreted to be erosional outliers or outcroppings of limestone from the Edisto Formation. General dimensions of the rock pinnacles are given in the table below.

Table B-8. NOAA diver surveyed rock pinnacle dimensions.

ID	Length	Width	Elevation	Type
Pinnacle 1	246 ft	6.5 ft	-39.2 MLLW	Limestone
Pinnacle 2	262 ft	9.8 ft	-42.8 MLLW	Limestone
Pinnacle 3	341 ft	49.2 ft	-41.5 MLLW	Limestone
Pinnacle 4	UNK	UNK	-42.8 MLLW	Limestone

5.2.4. Great Lakes Dock and Dredging Claim 1999.

The Great Lakes Dock and Dredging Company filed a Type-I differing site condition claim for reimbursement of additional costs associated with deepening the entrance channel in 1999. Great Lakes claimed that USACE did not properly characterize the rock within the entrance channel, which resulted in delays, fuel expenditures, and mobilization of additional equipment. GLDD claimed the rock was much stronger and more widespread than what was estimated by USACE.

Figure B-39. GLDD claim of excessively strong rock in the entrance channel.

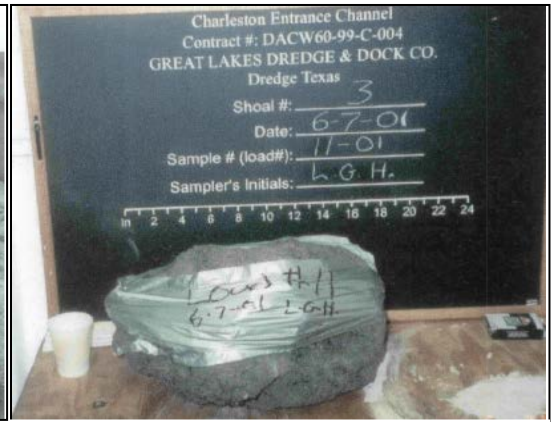


Figure B-37. GLDD limestone cobble selected for testing

GLDD conducted unconfined compressive strength testing of cobble-sized material encountered during dredging operations. The test results are presented below in Table B-9 and the general location for each grab sample is shown in Plate 2.

Table B-9. UCS data from the 1999 Great Lakes Docks and Dredging Type-I differing site condition claim.

Entrance Channel Stationing	Sample ID	Channel Range	Min UCS (psi)	Max UCS (psi)
655+00	217	-250	0	186
650+00	220	-250	246	293
649+00	221a	-157	0	93
610+00	223a	-70	124	182
598+00	226B	0	126	138
597+00	241A	270	112	145
596+00	227A	0	163	171
595+00	244A	270	225	955
591+00	246	270	137	257
590+00	234AB	0	114	1670
585+00	239A	0	419	453
584+00	036-01	0	0	547
583+00	001-01	270	0	248
581+00	003-01	270	0	458
580+00	039-01	0	0	313
580+00	29	-270	341	364
578+00	31	-270	0	417
575+00	045-01	0	0	214
574+00	012-01	270	0	374
574+00	36	-270	0	173
569+00	051-01	0	0	497

Entrance Channel Stationing	Sample ID	Channel Range	Min UCS (psi)	Max UCS (psi)
567+00	017-01	270	0	499
565+00	055-01	0	0	348
564+00	020-01	270	0	518
563+00	47	-270	0	255
560+00	059-01	0	0	211
558+00	024-01	270	0	487
555+00	063-01	0	0	416
554+00	54	-270	0	272
554+00	028-01	270	0	456
550+00	067-01	0	0	740
547+00	033-01	270	0	426
545+00	071-01	0	0	198
543+00	161-01	270	0	364
542+00	162-01	270	0	237
540+00	075-01	0	0	396
540+00	64	-270	0	87
538+00	167-01	270	0	964
535+00	079-01	0	0	210
533+00	68	-270	0	300
532+00	172-01	270	0	264
530+00	083A-01	0	230	429
528+00	72	-270	0	163
528+00	177-01	270	0	361
525+00	088-01	0	0	739
524+00	182-01	270	0	256
520+00	092-01	0	0	994
518+00	187-01	270	0	307
517+00	188-01	270	0	186
515+00	098-01	0	0	268
514+00	192-01	270	0	295
512+00	102-01	0	0	268
508+00	198-01	270	0	373
505+00	108-01	0	0	331
504+00	203-01	270	0	373
501+00	115-01	0	0	174
498+00	209-01	270	0	251
497+00	256-01	-270	0	253
495+00	121-01	0	0	555
493+00	260-01	-270	0	206
490+00	125-01	0	0	102
488+00	218-01	270	0	193
487+00	266-01	-270	0	463
485+00	131-01	0	0	377
483+00	224-01	270	0	769
483+00	272-01	-270	0	223
480+00	138-01	0	0	167
478+00	278-01	-270	0	273
478+00	231-01	270	0	286
475+00	147-01	0	0	201
473+00	238-01	270	0	352
472+00	2-5, 1A-1E	-270	263	535
471+00	281-01	-270	0	229
470+00	153-01	0	0	433
468+00	244-01	270	0	348
468+00	286AB-01	-270	171	289
466+00	158-01	0	0	188
464+00	293-01	-270	245	302
458+00	301-01	-270	0	195

Entrance Channel Stationing	Sample ID	Channel Range	Min UCS (psi)	Max UCS (psi)
453+00	309-01	-270	0	195
447+00	315-01	-270	0	175
445+00	318-01	-270	0	111

Charleston District’s Claims Board later indicated that the unconfined compressive strength tests that were supplied by GLDD do not represent the in-situ strength of the intact rock mass. Prominent engineering geologists (Hoek et al, 1995; Bieniawski, 1989, 1984, 1976, 1973; Romana, 1989; and Deere, 1964) indicate that features such as joint planes, fractures, fissures, and weak bedding planes control the overall strength of a rock mass, rather than the strength of individual pieces of rock. Comparing the GLDD data to the Rock Strength Category (Hoek et al, 1995), which is considered to be an industry standard, the Claims Board stated the following;

- 17 samples (16%) fell into the very weak category, with UCS < 180 psi
- 80 samples (76%) fell into the weak category, 181 psi < UCS < 725 psi
- 8 samples (7.6%) fell into the moderately weak category, 726 psi < UCS < 1812 psi.

The Charleston Claims Board concluded that 92% of the samples were weak to very weak rock and that there was no basis for GLDD claim of differing site conditions. Furthermore, the higher strength values presented by GLDD are still within the category of moderately weak rock, which can be effectively removed by dredging methods. These higher strength values should not be considered representative of the entire in-situ rock mass. These values are skewed to represent material that survived its travel intact through the cutter-head, dredge plant, pumps and piping, which were picked over and sampled for UCS strength testing. At best, these strength values represent the upper limits to the strength of the limestone, or some silicified horizon that was encountered during dredging operations.

Contrary to the recommendations of the Charleston Claims Board, it was determined that there was some merit to GLDD’s change of site condition claim. The claim was eventually settled by litigation, and GLDD was awarded approximately half of their original claim[17].

5.2.5. Geophysical Survey 2012.

It was determined early in 2012 that to properly characterize the strength of the limestone, USACE would need to collect additional core samples via drilling, and submit these samples to its own laboratory for strength testing. Prior to sample collection, USACE would need to locate where the bedrock crops out within the existing channel.

USACE, Charleston District contracted with the Center for Marine and Wetland Studies at Coastal Carolina University (CCU) in order to conduct a geophysical survey to delineate hardbottom habitats and map the top of bedrock within the Charleston Harbor Entrance Channel and other improvement areas. The geophysical methods used involved side-scan sonar, sub-bottom profiling, and magnetic mapping. CCU utilized the sub-bottom profiling to contour the seafloor, top of sediment, and top of rock surfaces. Of these three products, the top of bedrock

[17] The 1999 GLDD claim is believed to have been settled at 24 million dollars.

surface was considered the most important to the project because it was considered essential in developing a drilling exploration plan.

The geophysical mapping of the entrance channel was conducted from November, 2012 through January, 2013, using UNC-Wilmington's Research Vessel (R/V) *Cape Fear*. Equipment used included an EdgeTech sb512i CHIRP sub-bottom reflection sonar tow-fish with EdgeTech acquisition software. The CHIRP towfish is towed behind the vessel, where it emits an acoustic signal at a specified frequency, velocity and time interval. The instrument then "listens" for the return echo reflected back from the seafloor and underlying sediment. As the sound wave encounters and travels though different earth materials, the wave attenuates, and slows down before it is reflected back to the towfish receiver. The two-way travel time of the reflected signal is then recorded; minute differences in the two-way travel time indicate changing materials or lithology. Towfish navigation was obtained by a topside Northstar 965 DGPS receiver. The sub-bottom reflection profiles were acquired using a 0.5-8.0 kHz CHIRP signal with a 5-ms sweep, and were georeferenced in NAD 1983 South Carolina State Plane Feet. The CHIRP sub-bottom data was post-processed using SIOSEIS and Seismic Unix software packages, and corrected for ship heave, extraneous noise, tidal effect and vertical towfish superposition. The top of bedrock surface was digitized from the CHIRP sub-bottom profiles using Kingdom Suite Software. The surface was then gridded in accordance with USACE instructions for use with ESRI ArcGIS 9.3 software.

Figure B-38. EdgeTech CHIRP sonar towfish

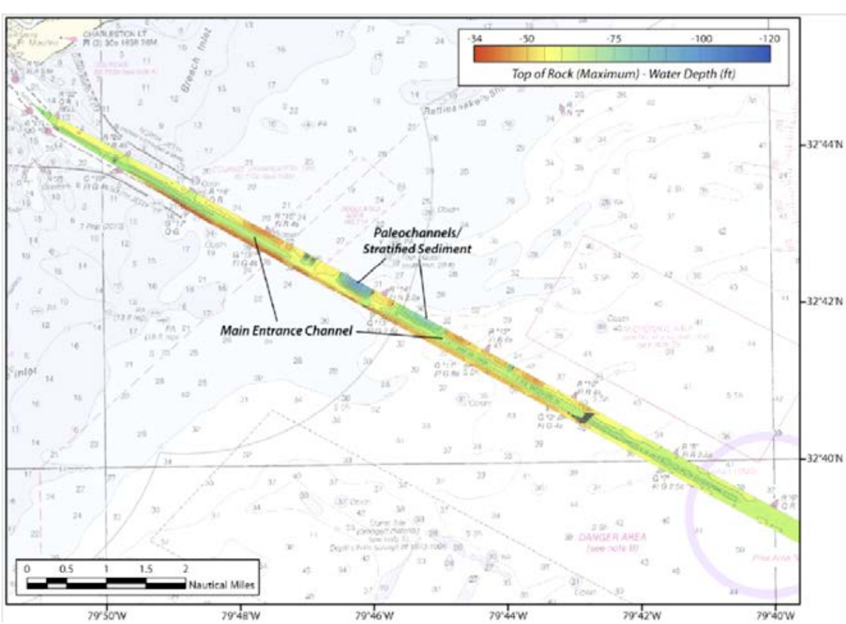


Figure B-39. CCU geophysical top of rock survey.

The CCU geophysical survey revealed that the seafloor morphology across the entrance channel consists of a series of NE/SW trending sediment ridges. The ridges were interpreted by CCU to be a feature resulting from the accumulation of modern surficial sediment. Where present, modern surficial sediment was found to be thin, transitory and patchy, often less than a foot thick; however, within the sediment ridges, the material is up to 10-15 feet thick.

Elsewhere, the sub-bottom was found to be homogeneous, featureless, essentially mimicking the bathymetric expression of the seafloor. These areas were interpreted to be representative of consolidated seafloor sediments, or exposed bedrock on the seafloor which was uncovered since the last dredge deepening. CCU identified several deep anomalies which are colored green to dark blue in Figure B-42. These anomalies are interpreted by CCU to be north-trending filled paleofluvial channels which are incised into the entrance channel. The in-fill material, based upon available drilling logs, consists of interbedded clayey and sandy material.

The CCU geophysical top of rock surface data was imported into ArcGIS by the Wilmington District, processed into a simpler TIN file, and re-contoured into 2-foot colored intervals for clarity, see Plate 4. The boring locations from the previous subsurface investigations are also plotted, with depth to top of rock and bottom of hole added for reference. Referencing the bathymetry in Plate 1 and the geophysical in Plate 4, there appears to be little difference between the bathymetric surface and geophysical top of rock in entrance channel segments EC-1, EC-3, and EC-9 through EC-14. Features resembling a series of buried, narrow to wide, paleo-fluvial channels (blue to deep blue color) are shown in entrance channel segments EC-4, EC-5, EC-6, EC-7 and EC-8. A broad geophysical top of rock high (red) is observed on the north side of EC-4 and in the middle of EC-5 between two buried paleo-fluvial channels (blue). The geophysical top of rock surface diverges significantly from the bathymetric surface in EC-15, and continues to deepen to depths greater than -70 feet MLLW out to EC-20. The staff at CCU suggested that this deepening may indicate subsidence, or possibly the presence of softer unconsolidated materials in the subsurface. A washprobe exploration program was deemed necessary by the PDT in order to ground truth the geophysical top of rock, and further constrain the drilling location for recovering representative samples of limestone for strength testing.

5.2.6. Washprobe Exploration Program, 2013.

Athena Technologies, Inc. (Athena) was contracted through the South Carolina Ports Authority (SCSPA) in February, 2013 to perform washprobing within the Charleston Harbor Entrance Channel, and the proposed 3-mile extension. The purpose of the washprobing effort was to ground-truth the geophysical survey conducted by CCU, and to better determine where there were substantial bodies of rock or consolidated material. The Wilmington District Geotechnical Section compared the CCU geophysical survey (Plate 4) with existing bathymetry, overlaying historical boring data (Plate 1), and geo-located GLDD claim data (Plates 2 & 3), to develop a comprehensive, yet prioritized washprobe target list. A listing of 301 washprobe targets was provided to Athena late February, 2012 for immediate contract execution. The washprobing effort involved the use of two vessels, the (R/V) *Artemis* and the fishing vessel (FV) *Miss Georgia*, in order to execute the contract in a timely manner. Athena contracted a larger third vessel, the FV *Miss Sandra II*, to provide a larger sampling platform and facilitate contract completion. The vessels navigated to each of the pre-designated washprobe locations using differential global positioning systems (DGPS),

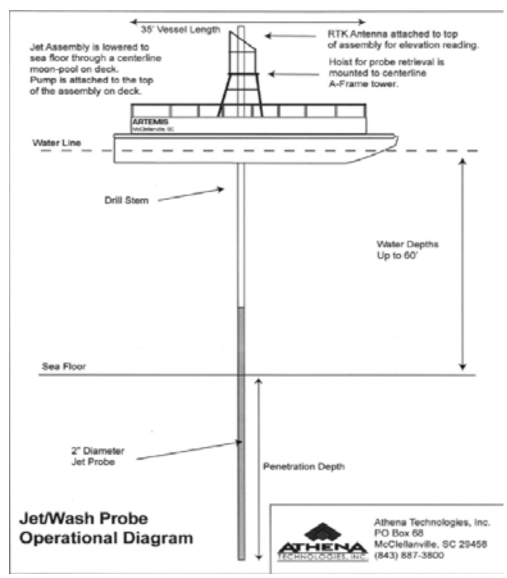


Figure B-43. Athena washprobe methodology schematic.

interfaced with HYPACK software. Once on-site, the vessels were immobilized via anchoring. The depth to the seafloor was determined either by lead line (in areas of low current velocity), by the length of jetting pipe (in areas of competent seafloor), or by fathometer in areas of high current velocity and soft seafloor material. Elevation was recorded using a Trimble R8 Global Navigation Satellite System receiver, which utilized the South Carolina Virtual Reference Station (VRS) as a base station. The elevation data was recorded in North American Vertical Datum of 1988 (NAVD 88) and later corrected to local Mean Lower Low Water using the National Oceanic and Atmospheric Administration's (NOAA) vertical datum transformation software VDatum (Version 3.2). Athena notes that use of the R8 GNSS receiver was limited to the range of cellular service, which they boosted to a maximum range of 15-miles using onboard signal amplification equipment.

The washprobes were advanced into the seafloor using a 1.5-inch hollow steel probe, 2-inch steel drill stems, and a 3-inch flexible hose connected to a water pump aboard the work vessel (Figure B-46 and Figure B-47). The probe, pipe and hose were connected via reducers and cam-lock pipe fittings. The operator would lower the washprobe, in sections, to the seafloor, at which point the water pump was turned on. The probe was then advanced until refusal was encountered. Upon refusal, the R8 GNSS was placed atop of the drill stem and the xyz data was recorded. Once complete, the probe was retrieved using a mechanical winch.


Figure B-44. Athena washprobing operation

Offshore weather became a constraining factor to the investigation. From the beginning of the washprobing effort, early spring time nor'easters produced hazardous wave conditions preventing Athena from conducting extensive washprobing. Due to prolonged bad weather and an aggressive project timeline, Athena only completed 194 of the 301 washprobes assigned. The washprobe data was submitted to Wilmington District for mapping and analysis. The washprobe locations and refusal depths were plotted atop existing bathymetry and historical boring data using ArcGIS software, shown in Plates 5 and 6. A summarized table of the washprobe results is provided in Table B-10. It was determined by the PDT that the 194 washprobes executed by Athena would suffice in aiding the geotechnical team's effort to target areas for future rock coring.

Table B-10. Summary results from the 2013 washprobe exploration, conducted by Athena Technologies.

Date	East (x)	North (y)	Ocean Bottom (MLLW)	Top of Refusal Elevation (feet relative to MLLW)	Thickness of Unconsolidated Sediment
3/29/13	2400093.63	306528.98	Undefined	-54.3	Undefined
3/29/13	2400959.67	306662.58	Undefined	-51.8	Undefined
3/29/13	2400884.56	306120.70	Undefined	-56.8	Undefined
3/29/13	2401136.44	306243.05	Undefined	-52.5	Undefined
3/29/13	2402175.93	305993.47	Undefined	-57.4	Undefined
3/29/13	2401843.70	305887.18	Undefined	-53.0	Undefined
3/28/13	2401458.21	305815.28	Undefined	-56.1	Undefined
3/28/13	2402456.41	305849.19	Undefined	-54.0	Undefined

Date	East (x)	North (y)	Ocean Bottom (MLLW)	Top of Refusal Elevation (feet relative to MLLW)	Thickness of Unconsolidated Sediment
3/28/13	2402696.73	305419.92	Undefined	-53.5	Undefined
3/28/13	2402877.25	305038.58	Undefined	-53.5	Undefined
3/28/13	2403801.43	305249.22	Undefined	-54.0	Undefined
3/28/13	2404042.79	304812.00	Undefined	-53.6	Undefined
3/28/13	2404158.33	304346.90	Undefined	-53.6	Undefined
3/28/13	2405341.95	304395.49	Undefined	-57.1	Undefined
3/28/13	2403580.57	304469.26	Undefined	-48.9	Undefined
3/27/13	2406103.71	303521.91	Undefined	-53.3	Undefined
3/27/13	2406686.38	303506.27	Undefined	-54.2	Undefined
3/28/13	2405807.42	303370.58	Undefined	-54.4	Undefined
3/27/13	2407717.98	302853.53	Undefined	-52.7	Undefined
3/27/13	2408023.45	302410.44	Undefined	-51.3	Undefined
3/27/13	2409893.23	301557.18	Undefined	-50.6	Undefined
3/27/13	2409431.91	301814.01	Undefined	-49.7	Undefined
3/27/13	2408492.05	302359.96	Undefined	-52.9	Undefined
3/19/13	2410359.58	301295.65	Undefined	-50.1	Undefined
2/21/13	2410975.27	300727.50	Undefined	-51.0	Undefined
3/27/13	2411863.23	300809.30	Undefined	-51.8	Undefined
3/27/13	2412054.81	300390.80	Undefined	-53.1	Undefined
3/27/13	2412409.86	300245.23	Undefined	-50.3	Undefined
3/19/13	2415692.18	298058.31	Undefined	-55.7	Undefined
4/7/13	2435939.97	287445.58	-41.9	-63.6	21.7
4/8/13	2429009.20	291401.86	-51.2	-65.9	14.7
4/8/13	2428618.35	291271.72	-52.3	-66.3	14.0
4/8/13	2431347.64	289789.79	-50.8	-64.8	14.0
4/8/13	2428193.41	291145.44	-52.2	-66.2	14.0
4/8/13	2430800.21	289695.79	-51.2	-65.0	13.8
4/7/13	2435373.30	287697.28	-50.9	-64.6	13.7
4/8/13	2432041.78	289797.46	-51.1	-64.4	13.3
4/7/13	2435667.67	287541.87	-50.9	-63.8	12.9
4/8/13	2423629.24	294472.66	-52.1	-64.4	12.3
4/8/13	2432709.49	288781.67	-54.8	-67.0	12.2
4/8/13	2421425.53	295005.47	-51.8	-63.8	12.0
3/2/13	2372041.37	321849.80	-42.7	-54.7	12.0
4/13/13	2374620.81	321379.45	-42.2	-53.8	11.6
4/13/13	2374830.25	321145.18	-46.2	-57.7	11.5
4/16/13	2425350.56	292849.04	-51.3	-62.8	11.5
4/8/13	2426469.29	292070.92	-52.0	-63.4	11.4
4/8/13	2432987.85	288830.96	-55.5	-66.7	11.2
4/7/13	2437855.26	285745.71	-52.9	-63.9	11.0
4/8/13	2422542.62	294231.22	-52.7	-63.4	10.7
4/8/13	2423032.56	294359.58	-52.6	-63.3	10.7
4/8/13	2433514.50	288829.00	-55.3	-65.8	10.5
4/7/13	2436228.05	287620.87	-51.5	-62.0	10.5
4/8/13	2392010.76	311874.39	-40.0	-50.5	10.5
4/8/13	2421671.89	295412.54	-51.5	-61.9	10.4
4/7/13	2439074.17	285532.48	-51.4	-61.7	10.3
4/8/13	2427257.54	292357.95	-51.2	-61.4	10.2
4/8/13	2425427.96	292960.56	-52.1	-62.3	10.2
4/9/13	2378179.48	318506.55	-45.1	-55.2	10.1
4/7/13	2443333.22	283745.52	-53.0	-63.1	10.1
4/9/13	2385782.67	315275.06	-38.3	-48.3	10.0
4/8/13	2426825.96	292202.68	-52.0	-61.8	9.8
4/7/13	2440327.64	285502.21	-51.5	-61.3	9.8
4/7/13	2442753.74	283746.53	-53.0	-62.6	9.6
4/7/13	2442387.69	283386.08	-52.8	-62.3	9.5
4/9/13	2382137.36	316571.47	-51.4	-60.7	9.3

Date	East (x)	North (y)	Ocean Bottom (MLLW)	Top of Refusal Elevation (feet relative to MLLW)	Thickness of Unconsolidated Sediment
4/10/13	2375336.56	319895.90	-41.8	-50.8	9.0
4/14/13	2372668.56	321571.54	-46.9	-55.8	8.9
4/14/13	2364258.45	326543.99	-51.3	-60.2	8.9
4/16/13	2422693.91	294222.96	-51.5	-60.4	8.9
4/7/13	2441646.85	283819.87	-53.5	-62.2	8.7
4/9/13	2377464.61	318866.65	-44.7	-53.2	8.5
4/7/13	2443010.87	283288.18	-54.7	-63.1	8.4
4/16/13	2422487.23	295059.06	-51.4	-59.7	8.3
4/14/13	2373025.41	322129.28	-47.2	-55.3	8.1
4/7/13	2447343.35	281580.52	-54.2	-62.1	7.9
4/14/13	2363823.06	326677.27	-50.3	-58.0	7.7
4/9/13	2383492.06	316084.10	-51.9	-59.5	7.6
4/8/13	2391313.47	312254.13	-42.6	-50.0	7.4
4/14/13	2374345.44	321377.24	-47.2	-54.5	7.3
4/16/13	2423375.98	294413.98	-52.2	-59.4	7.2
4/10/13	2356789.78	331018.90	-48.7	-55.8	7.1
4/16/13	2419711.93	296479.27	-51.1	-58.2	7.1
4/14/13	2375877.17	320495.92	-48.1	-55.1	7.0
4/10/13	2378503.05	319145.34	-48.2	-54.9	6.7
4/14/13	2374554.10	320523.62	-47.1	-53.6	6.5
4/16/13	2420333.03	295935.80	-52.1	-58.6	6.5
4/8/13	2420194.85	295712.82	-52.6	-58.9	6.3
4/8/13	2422118.49	294848.54	-51.8	-57.8	6.0
4/14/13	2372973.69	321359.57	-46.5	-52.5	6.0
3/2/13	2372361.65	322212.88	-48.1	-53.9	5.8
4/18/13	2376924.31	319175.67	-48.1	-53.8	5.7
4/14/13	2373472.89	321173.00	-48.0	-53.1	5.1
4/10/13	2379734.06	318577.53	-45.7	-50.7	5.0
4/17/13	2372652.93	321688.48	-49.6	-54.4	4.8
4/17/13	2370678.76	322514.12	-45.1	-49.8	4.7
4/10/13	2375926.04	319732.48	-49.5	-54.2	4.7
4/16/13	2416131.82	298037.89	-51.0	-55.6	4.6
4/14/13	2361322.75	327635.40	-49.3	-53.7	4.4
4/10/13	2358430.31	330041.08	-51.4	-55.6	4.2
4/9/13	2416793.61	298206.74	-52.1	-56.0	3.9
4/9/13	2418857.17	296998.87	-53.4	-57.1	3.7
4/18/13	2360158.65	329086.73	-50.6	-54.3	3.7
4/17/13	2373102.72	321884.48	-50.0	-53.7	3.7
4/10/13	2375319.36	320317.55	-51.3	-54.9	3.6
4/16/13	2419235.69	296044.72	-52.8	-56.4	3.6
4/17/13	2370187.21	323034.65	-49.8	-53.0	3.2
4/10/13	2377678.31	319655.52	-46.9	-49.9	3.0
4/9/13	2382855.22	316428.99	-51.9	-54.8	2.9
4/10/13	2380290.54	318123.99	-51.2	-53.9	2.7
4/18/13	2389010.17	312470.55	-42.8	-45.5	2.7
4/18/13	2388035.03	314022.60	-46.2	-48.9	2.7
4/9/13	2381701.36	316794.90	-51.3	-53.8	2.5
4/14/13	2361816.88	328285.52	-51.4	-53.9	2.5
4/9/13	2417541.35	296966.62	-51.7	-54.2	2.5
4/13/13	2370130.21	323824.56	-49.3	-51.6	2.3
4/18/13	2376127.91	320120.35	-52.7	-55.0	2.3
4/16/13	2417779.50	297399.23	-52.2	-54.4	2.2
4/9/13	2419335.50	296413.08	-52.2	-54.4	2.2
3/15/13	2382993.54	316022.32	-49.5	-51.6	2.1
4/3/13	2396968.45	308853.48	-54.1	-56.1	2.0
4/16/13	2414695.95	298541.88	-50.8	-52.8	2.0
4/9/13	2379940.40	317537.64	-52.8	-54.7	1.9

Date	East (x)	North (y)	Ocean Bottom (MLLW)	Top of Refusal Elevation (feet relative to MLLW)	Thickness of Unconsolidated Sediment
4/17/13	2390988.52	312110.45	-50.8	-52.6	1.8
4/14/13	2370877.20	323018.45	-51.6	-53.4	1.8
4/7/13	2415442.44	298463.55	-55.8	-57.2	1.4
4/17/13	2374970.56	320620.91	-52.5	-53.9	1.4
4/8/13	2395720.80	308747.45	-49.5	-50.8	1.3
4/10/13	2356544.21	330385.67	-51.7	-53.0	1.3
4/13/13	2369105.55	323965.82	-50.5	-51.8	1.3
4/18/13	2375720.90	320089.11	-51.0	-52.3	1.3
4/9/13	2379517.02	317750.44	-52.9	-54.1	1.2
4/13/13	2366731.90	325703.59	-49.0	-50.2	1.2
4/17/13	2391760.39	311632.01	-51.0	-52.2	1.2
4/2/13	2400176.23	307240.98	-51.5	-52.7	1.2
4/17/13	2394325.40	309642.92	-50.8	-51.9	1.1
4/9/13	2385470.81	315142.86	-51.9	-52.8	0.9
4/10/13	2375689.15	320377.52	-51.9	-52.8	0.9
4/14/13	2364958.65	326627.38	-50.8	-51.7	0.9
4/13/13	2368830.51	324537.45	-47.5	-48.3	0.8
4/17/13	2394117.84	310158.77	-51.3	-52.1	0.8
4/14/13	2365713.31	325333.81	-50.2	-51.0	0.8
4/8/13	2387866.99	313461.56	-56.2	-57.0	0.8
4/10/13	2359691.64	328854.52	-49.3	-50.1	0.8
4/8/13	2388882.85	312638.96	-55.0	-55.7	0.7
4/17/13	2373349.32	321520.86	-52.4	-53.1	0.7
4/17/13	2403361.32	305181.50	-50.8	-51.5	0.7
4/10/13	2360013.71	329031.26	-53.0	-53.6	0.6
4/10/13	2358662.38	329029.29	-50.0	-50.5	0.5
4/8/13	2391025.35	311643.85	-55.6	-56.1	0.5
4/17/13	2368084.43	324515.99	-53.8	-54.3	0.5
4/9/13	2386770.20	313981.98	-51.3	-51.7	0.4
4/9/13	2380139.41	317732.77	-51.3	-51.7	0.4
4/14/13	2367598.45	324340.15	-50.6	-51.0	0.4
4/17/13	2413501.94	299343.88	-49.8	-50.2	0.4
4/10/13	2376828.29	319590.17	-53.0	-53.3	0.3
4/17/13	2378343.17	318738.93	-53.0	-53.0	0.0
4/16/13	2386182.25	314726.46	-56.0	-56.0	0.0
4/17/13	2395706.24	309122.64	-51.1	-51.1	0.0
4/17/13	2396211.48	308510.28	-46.3	-46.3	0.0
4/17/13	2404258.04	304099.98	-48.4	-48.4	0.0
4/17/13	2407572.73	302602.64	-49.0	-49.0	0.0
4/17/13	2408441.42	302126.91	-49.3	-49.3	0.0
4/17/13	2411744.78	300488.78	-49.8	-49.8	0.0
4/17/13	2412582.26	300331.67	-49.0	-49.0	0.0
4/9/13	2382763.35	316890.97	-48.5	-48.4	-0.1
4/2/13	2400074.03	306863.25	-54.5	-54.3	-0.2
4/16/13	2384266.29	315993.52	-51.4	-51.2	-0.2
4/14/13	2373678.54	321384.71	-52.6	-52.3	-0.3
4/7/13	2415048.62	298829.78	-54.7	-54.0	-0.7
4/3/13	2397884.42	308061.49	-54.8	-54.1	-0.7
4/8/13	2396167.46	309058.31	-55.1	-54.3	-0.8
4/8/13	2395740.50	309605.26	-55.3	-54.4	-0.9
4/8/13	2396625.52	309137.83	-54.5	-53.4	-1.1
4/7/13	2414313.93	298876.34	-53.8	-52.7	-1.1
4/9/13	2382256.82	316263.82	-52.8	-51.6	-1.2
4/8/13	2392701.92	310714.95	-56.8	-55.6	-1.2
4/8/13	2392176.07	310894.24	-57.3	-56.0	-1.3
4/8/13	2394040.47	310454.78	-56.1	-54.4	-1.7
4/3/13	2397052.52	308528.20	-55.1	-53.3	-1.8

Date	East (x)	North (y)	Ocean Bottom (MLLW)	Top of Refusal Elevation (feet relative to MLLW)	Thickness of Unconsolidated Sediment
4/9/13	2387230.61	314152.31	-54.1	-52.2	-1.9
4/7/13	2412962.32	300034.56	-52.8	-50.8	-2.0
4/7/13	2415985.85	298619.96	-57.0	-54.7	-2.3
4/7/13	2414523.98	299112.31	-54.3	-51.7	-2.6
4/2/13	2398853.13	307566.18	-56.9	-54.2	-2.7
4/17/13	2366790.81	325190.22	-55.9	-53.1	-2.8
4/7/13	2413839.52	299515.39	-54.0	-51.2	-2.8
4/16/13	2382199.71	316943.98	-56.0	-53.2	-2.8
4/16/13	2386257.85	314109.39	-56.4	-53.5	-2.9
4/17/13	2379889.30	317855.62	-55.3	-52.1	-3.2
4/17/13	2381133.77	317257.60	-54.7	-51.2	-3.5
4/9/13	2387596.76	313960.72	-57.0	-53.3	-3.7
4/16/13	2383941.29	315839.18	-56.0	-51.7	-4.3
4/8/13	2389264.04	313054.16	-56.3	-51.6	-4.7
4/9/13	2385561.73	314367.99	-56.2	-50.4	-5.8

Referencing Plates 4 and 5, the results of the washprobe exploration indicated that much of the new work material within proposed dredge prism of EC-1 through EC-2 is likely unconsolidated material. Shallow subcroppings of limestone were encountered near the boundaries between channel segments EC-1 and EC-2, and EC-2 with EC-3. Washprobe refusal indicates that segments EC-3 to EC-5 are floored by consolidated material between elevations -51 to -54 feet MLLW. This refusal surface appears to become more varied in EC-6 through E-9, as it ranges from -50.4 to -60.7 feet MLLW. Within channel segments EC-10 through EC-13, washprobe refusal occurred directly on channel bottom. This reflects the harder nature of the bedrock that was exposed during the last dredge deepening; little to no unconsolidated sediment is present. The washprobes in entrance channel segments EC-14 to EC-16 indicate the presence of more substantial amounts of unconsolidated sediments that appears to thicken seaward. This sediment becomes 10-feet or greater in thickness in EC-17. The northeast-southwest trending ridges located in the outer channel reaches (mentioned in Section 6.1.4. “Entrance Channel Existing Conditions”) are comprised of unconsolidated sediment based upon washprobe data. Within entrance channel segments EC-18 to EC-21, the washprobes were advanced to -60 feet MLLW without encountering refusal or competent material.

In summary, the washprobe exploration revealed that

- Entrance channel segments EC-2 though EC-14 likely contained rock within the proposed new work dredge prism. Segments EC-4 through EC-5 and EC-10 through EC-12, appear to have the most significant amounts of shallow rock in the subsurface.
- Entrance channel segments EC-15 though EC-21 appears to have significant amounts of unconsolidated sediments, with very little hard material. Washprobe refusal, where encountered, was well below the proposed dredge prism.
- The shoal or ridgelines within the proposed 3-mile extension is likely composed of a thick (>10 feet) blanket of unconsolidated loose-soft material because no refusal was encountered during washprobing. Washprobe penetration was > -60 feet MLLW.

5.3 Rock Core Target Refinement

The PDT decided to conduct the rock coring, sampling and testing during the feasibility phase, FY-2013 because of the cost-share partnership with SCSPA, project timeline, and the availability of low-density, high-demand assets such as drilling crews and jack-up vessels to do the work. Wilmington District estimated that a total of 55 borings with the requisite testing could be completed, given the financial resources were available. With a 55-boring constraint, Wilmington District developed a rock coring plan, using an iterative "targeting" process that prioritized the rock cores based upon several factors;

- Previous occurrence of limestone bedrock in historical borings or mapped on seafloor
- Percentage of limestone bedrock in each channel segment, ascertained earlier in the year
- Volume of new work material
- SPT and UCS values of rock and soil for each channel segment
- Geophysical top of rock

The Wilmington District Geotechnical Section re-investigated the percentage of unconsolidated, soft rock, and competent rock for each of the entrance channel segments for the purpose of prioritizing where to focus drilling and sampling efforts. Table B-11 below summarizes the results of the analysis. The new work volumes were calculated by Charleston District (SAC) for each of the channel segments at two (then-proposed design depths) depths proposed by the PDT, -55 and -58 feet MLLW. The segments are sorted by maximum volume, which is then compared to the percentage of type material (based on historical borings). Lastly, these are compared against the average depth of geophysical top of rock and washprobe refusal. Each entrance channel segment was then color coded for high, moderate, or low probability of having substantial amounts of rock within the dredging prism.

Table B-11.Probability matrix for encountering rock based upon historical data.

Estimated Entrance Channel New Work Quantities (CY)				% MATERIAL IN DREDGE CUT TO -58 MLLW				Average Depth TOR (Geophysical)	Average Depth TOR (Washprobe)	Probabilty of Encountering Bedrock
	% (58') QTY	55'	58'	Avg % Uncon	Avg % SoftRock	Avg % CompRock	% Unknown			
Segment 4	8.4%	402,897	737,540	35%	52%	0%	14%	-52	-53.3	HIGH
Segment 5	8.3%	401,301	729,419	46%	34%	11%	9%	-46	-53.4	HIGH
Segment 6	7.4%	349,131	652,831	52%	38%	0%	10%	-48	-53.2	MODERATE
Segment 3	7.1%	306,321	625,978	59%	7%	0%	34%	-54	-52.8	LOW-MODERATE
Segment 7	6.5%	285,333	573,134	62%	33%	0%	5%	-50	-52.9	HIGH
Segment 1	6.5%	265,711	569,596	76%	0%	0%	24%	-54	-53.7	LOW
Segment 10	6.3%	272,282	550,547	30%	16%	47%	7%	-54	-53.3	HIGH
Segment 11	5.9%	250,045	517,333	17%	5%	73%	5%	-52	-53.4	HIGH
Segment 8	5.8%	252,198	507,662	54%	35%	6%	5%	-52	-52.1	HIGH
Segment 9	5.4%	227,373	476,307	38%	24%	34%	3%	-52	-52.8	HIGH
Segment 12	5.1%	198,198	450,290	18%	30%	52%	0%	-50	-51.6	HIGH
Segment 2	5.0%	159,265	435,529	58%	17%	5%	19%	-54	-53.9	MODERATE
Segment 13	4.9%	191,720	430,406	17%	33%	50%	0%	-48	-51.5	HIGH
Segment 16	4.2%	161,390	367,736	35%	31%	28%	6%	-58	-63.8	LOW
Segment 15	3.3%	121,885	289,292	0%	0%	0%	100%	-58	-60.5	LOW
Segment 14	3.3%	120,112	287,713	0%	0%	0%	100%	-52	-55.3	MODERATE
Segment 17	2.2%	70,524	188,858	0%	0%	0%	100%	-56	-65.5	LOW
Segment 19	1.7%	38,774	147,116	0%	0%	0%	100%	-60	-62.3	LOW
Segment 18	1.4%	28,801	118,868	0%	0%	0%	100%	-60	-64.0	LOW
Segment 20	1.2%	12,428	108,614	0%	0%	0%	100%	-65	-62.7	LOW
Segment 21	0.0%	21	2,470	0%	0%	0%	100%	-62	No Data	LOW
Total QTY (CY)	100.0%	4,115,709	8,767,238							

Rock core target selection was conducted using combined historical exploration overlays, which are provided in Plates 7 through 11. All historical borings, mapped subcroppings, GLDD data, geophysical and bathymetry were aligned and superimposed using ArcGIS software. Maximum rock strength and SPT N-values were then plotted against the centerline of the channel to enable targeted drilling of rock cores in areas having high probability of bedrock. An initial list of 120 potential targets was narrowed down to a final target list of 55-borings, which is based upon the historical occurrence of rock, geophysical data, probability (see Table B-11) and estimated volume of material per channel section (Plates 6 through 10). The boring plan was submitted to the PDT for approval mid July 2013, and was approved by both SAC and South Atlantic Division (SAD) soon thereafter.

The final drilling plan that was approved by the PDT is shown in Table B-12. The majority of the borings are located in EC-4 to EC-5 and in EC-10 to EC-12. No borings were placed in segments EC-1 or EC-15 though EC-21 due to the low probability of encountering bedrock.

Table B-12. Summary of 2013 rock core drilling plan approved by the PDT.

Segment	# Borings	Max Drill Depth	Est. TOR (MLLW)	Expected (Historical) Strata Type
EC-2	3	Continuously sample or (upon visual id rock) rock core to -60 feet, MLLW	≈ 52.5’	Stiff silty clay
EC-3	2		≈ 49.0’	Clayey sand & limestone
EC-4	5		≈ 53.0’	Clay, clayey sand & limestone
EC-5	5		≈ 54.0’	Limestone & dense silty sands
EC-6	5		≈ 53.0’	Limestone, loose clayey sands & clayey silt
EC-7	4		≈ 52.0’	Clays, clayey sands, little limestone
EC-8	4		≈ 55.0’	Limestone, dense silty sands & clays
EC-9	3		≈ 52.0’	Dense silty sand & limestone lenses.
EC-10	6		≈ 53.0’	Limestone & dense cemented sands
EC-11	6		≈ 53.5’	Limestone & dense silty sands
EC-12	6		≈ 52.5’	Limestone & dense cemented sand
EC-13	4		≈ 51.5’	Dense silty sand
EC-14	2		≈ 56.0’	Unknown

5.4 Field and Laboratory Methods

5.4.1 Offshore Drilling Program

The drilling program that was developed for the project was the result of a close partnership between USACE and SCSPA. Charleston District provided managerial, legal and administrative support. Wilmington District developed the scope, drilling plan, operational coordination, and project geologist. Savannah District provided the drilling equipment, experienced drill crews, and one of their geologists. The successful completion of the drilling program is due to the support provided from the SCSPA, which provided effective and efficient contracting services, and support facilities.

Figure B-40. SAS Failing 1500 Drilling Rig

The following chapter describes the equipment, field methods, and laboratory test methods used during the course of this investigation.

5.4.1.1. Drilling Rig & Floating Plant. The drilling rig used for this project was a gasoline powered Failing Model 1500. The drilling rig was built in the late 1970's, which features a retractable 32-foot tubular steel mast, mechanical clutch system, cable-reel draw works, a 5 x 6.5 inch Gardner Denver Pump system, and a 140-lb free falling weight for SPT sampling (Figure B-45). The Failing Model 1500 is capable of drilling 10-inch diameter borings to depths greater than 500-feet deep. The drilling rig is mounted on the bed of a heavy-weight dual axel diesel truck chassis for conventional land-based drilling. However, for offshore drilling operations, the drilling rig was removed and placed on a fabricated steel mount aboard ship.

Figure B-41. Precon Marine's W/V Cap'n Ray, jack-up vessel.

The floating plant used for the project was the Work Vessel (W/V) *Cap'n Ray*, owned and operated by Precon Marine, Inc (Figure B-46). The vessel hull dimensions are 64-feet long x 32-feet wide x 7 feet high (not including galley, sleeping areas, and pilot house). The vessel is powered by two 350 HP 8-71 Detroit diesel engines that can sustain a cruising speed of 3.5-4 mph. The *Cap'n Ray* can elevate itself a maximum of 66-feet above the seafloor using 3x 98-foot long spuds that are geared to 3-independently operated, hydraulic rack and pinion-type jacking systems. The vessel has an effective working area of 35- x 28-feet, a 12-inch diameter moon pool for drilling, 50kw electric generating capacity, and a 15-ton service lift crane with 70-foot boom. The Cap'n Ray was one of the only jack-up vessels of its type available for charter during the exploration timeframe. Similar vessels are presently in high-demand and must be chartered 2-years in advance of operations. The Cap'n Ray is based out of Hampton Roads, Virginia.

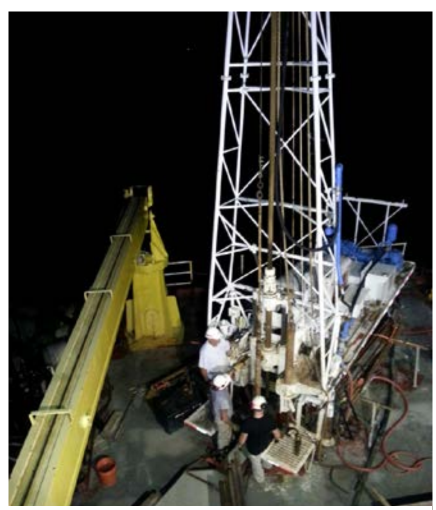


Figure B-47. SAS & SAW conducts 24-hour drilling operations aboard the Cap'n Ray mid-August 2013.

5.4.1.2. Drilling Operations. The drilling plan consisted of drilling a total of 55-borings within the entrance channel of Charleston Harbor, to a minimum elevation of -60 feet MLLW using the mud rotary method. The borings were established by first advancing 8-inch diameter steel casing to the seafloor. The ocean bottom was sounded through the inside of the casing using a weighted line, in order to avoid drag from the channel current. After the initial sounding, the first 18-inch Standard Penetrometer Test (SPT) drive was conducted. The casing was then advanced a short distance (< 1-foot) until mud circulation was established. The borehole was then continuously sampled using the SPT method (ASTM 1586) until the geologist visually determined that limestone bedrock had been encountered. At such point, the driller would switch over to rock coring methods to advance the boring to the completion elevation. The cost of conducting the exploration was estimated to be $980,917.00 for 25 days worth of drilling, or $25,737.00 per day of drilling[18]. Over half of the exploration budget was allocated to mobilizing and renting the *Cap'n Ray* with its accompanying crew. Therefore, the PDT decided it would be most cost effective to conduct 24-hour drilling operations (Figure B-47) in order to minimize rental days, increase drilling/productivity time, and minimize the potential for inclement weather to delay the drilling. Savannah District mobilized two drilling crews to the project; a day and night shift each consisting of a senior driller, and two helper/assistant drillers. SAS also mobilized a geologist to work with the day shift drilling crew. Wilmington District sent the project geologist, who worked with the night crew, and coordinated on-site drilling operations, such as movement order, schedule, ship-to-shore shuttling, and SAC-SCSPA communications on a 24-hour basis. Each shift worked approximately 12-hours; 0600 to 1800, and 1800 to 0600, with 1 hour allocated for ship to shore shuttling. The SCSPA contracted TowBoat U.S. to handle the daily ship-to-shore shuttle service.

The Cap'n Ray disembarked from the Precon Marine, Inc. shipyard in Norfolk, Virginia 05AUG13 and arrived in the Port of Charleston on the evening of 08AUG13. Savannah District mobilized its drilling crews early morning on 09AUG13 and arrived in Charleston later that morning. Upon arrival, the SAS drill crews unloaded equipment and cut the drilling rig from the truck and installed it onto a prefabricated mount that was then welded to the deck of the *Cap'n Ray*. Day-time drilling operations commenced on 10AUG13, with 24-hour operations coming into effect on 12AUG13. Once established, an average of 3-4 borings was drilled during each 24-hour period of operations. The borings were advanced to average completion elevation of -62 feet MLLW. On 30AUG13, the Cap'n Ray suffered a mechanical breakdown in one of the starboard hydraulic lift motors that raises/lowers the starboard spud. Drilling operations were placed on standby, having 49 out of 55 borings complete, while the ship's captain and crew initiated troubleshooting and repairs. Drilling operations were terminated on 02SEP13, when Precon Marine personnel determined that the damage to the starboard spud and hydraulic system was irreparable onsite, and would require the services of an experienced shipyard. Having completed 49 out of 55 borings, the PDT determined that despite losing 6 borings to a mechanical breakdown, overall, the exploration mission goals had been accomplished.

5.4.1.3. Horizontal Control. The horizontal location of each borehole was determined in the field using the HYPACK system installed aboard the Cap'n Ray. All horizontal data is referenced to South Carolina State Plane International Feet, NAD 1983. Precon Marine was given the coordinates for each of the proposed borings, which were then loaded into HYPACK. Horizontal control was very well established for each boring because the vessel's GPS receiver is georeferenced to the vessel's moon pool.

5.4.1.4. Vertical Control. Vertical elevation control was established in the field using two real-time kinematic (RTK) differential GPS transceivers, the Trimble 5700 and Trimble 5800. The elevation data was recorded in NAVD88 by the systems, with automatically corrected for tidal effects. An elevation datum transformation factor was then applied by hand to correct to MLLW elevation. The elevation was then applied to the top of the casing and to the seafloor sounding in order to determine elevation of the subsurface stratum.

[18] Based upon July 2013 SAW drilling cost estimate which used quoted rental, service, and labor rates.

5.4.1.5. Standard Penetrometer Test. The Standard Penetration Test (SPT) is described in ASTM D1586-08a as a test procedure by which a splitspoon sampler is driven, using a known energy, to obtain a representative soil sample for identification purposes, and to measure the resistance of the soil to penetration (compactness). The test provides an indication of the relative density of granular soils, such as sand and gravel. Soil strength parameters derived from the test are generally considered approximate, but they are deemed acceptable given the widespread use of the method and it's relatively low cost. Correlation between the blow-count (or N-value) and soil strength properties tends to be greater in sandy soils than in clayey soils. Despite this, the test method is used extensively to quantify soil properties for geotechnical engineering design.

Within the Standard Penetration Test, the compactness of the soil is chiefly determined by the degree to which the material adheres to the inner and outer surfaces of the splitspoon. The resultant friction resistance in soils to penetration is governed by the soil type, which was formalized by Terzaghi and Peck (1967). A general relationship exists between the soil compactness, SPT N-value, and the soil sample's resistance to penetration as shown in following table from Terzaghi and Peck (1967).

Table B-13. Relationship between SPT N-value and soils from Terzahi & Peck (1967).

Soil Type	**Soil Condition**	**SPT N-Value**	**Resistance Pressure /Unconfined Compressive Strength (psi)**	**Relative Density**	**Torvane Cohesion (psi)**
Granular Soils (Sand)	Very Loose	< 4	363 psi	0.15	---
	Loose	4-10	363-725 psi	0.15-0.35	---
	Medium Dense	10-30	725-1450 psi	0.35-0.65	---
	Dense	30-50	1450-2900 psi	0.65-0.85	---
	Very Dense	> 50	> 2900 psi	0.85	---
Fine-grained Soils (Silt/Clay)	Very Soft	< 2	4 psi	---	1.9 psi
	Soft	2-4	4-7 psi	---	1.9-3.6 psi
	Plastic	4-8	7-15 psi	---	3.6-7.3 psi
	Stiff	8-15	15-29 psi	---	7.3-14.5 psi
	Very Stiff	15-30	29-58 psi	---	14.5-29.0 psi
	Hard	> 30	> 58 psi	---	> 29.0 psi

The SPT procedure, as described in ASTM D1586-08, involves driving a standard thin-walled, 24-inch long, 2-inch OD/1-3/8-inch ID, splitspoon sampler a total depth of 18-inches into undisturbed soil. The driving energy for is imparted to the sampler (and length of drill rod) from the blows of a 140-lb hammer free-falling 30-inches. The number of blows to drive the sampler in three 6-inch increments is recorded. The first 6-inches of penetration is considered to be the seating drive. The sum of the number of blows required for the second and third 6-inches of penetration is termed the "standard penetration resistance" or the "N-value". The blows are applied and counted for each of the 6-inches until 18-inches of penetration is achieved. The test is terminated if; a total of 50- blows have been applied during any one of the three 6-inch increments, a total of 100-blows have been applied, or there is no observable advance in the sampler during the application of 10 successive blows of the hammer.

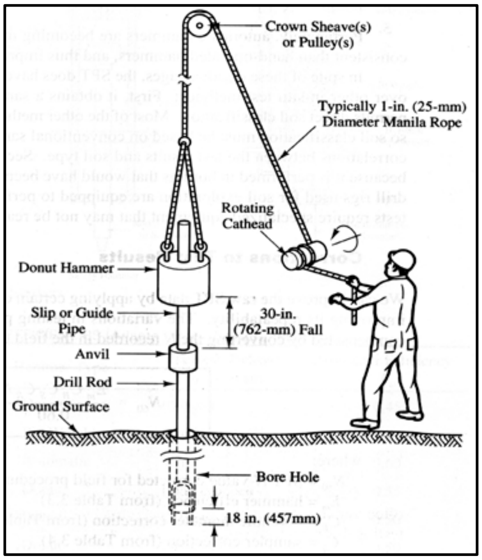


Figure B-42. Concept drawing of SPT method.

Mud rotary drilling procedures were used to advance the boring to the sampling depth. The drill bit used was a 5-1/2 inch fishtail bit having upward discharge, which facilitates cutting removal without disturbing the underlying strata to be sampled. Upon completion of the 18-inch drive, the splitspoon sampler was removed and the sample was logged by the geologist. The driller then drilled through the 18-inches that was just sampled and cleaned out the boring for the next SPT drive. The geologist was responsible for determining when the drilling procedure was to be changed to rock coring. This was based upon the occurrence of limestone, limestone gravels, or well-cemented material in the splitspoon sampler. The practice of using 50 blow count/6-in of SPT drive is not a good indicator within the study area because much of the limestone bedrock is soft or well indurated, and it disintegrates into sand and gravel sized particles during SPT sampling. Many of the historical borings that describe the presence of dense calcareous sand and gravel may have actually been limestone that was disintegrated during sampling.

5.4.1.6. Rock Coring. Rock coring was conducted in accordance with the guidelines established in EM 1110-1-1804 *"Geotechnical Investigations"* and ASTM D2113. Both HQ- and PQ-size double barrel, internally lined, wire line systems, with diamond impregnated core bits were used because of their superior sample retention capabilities in soft bedrock and cemented sands. The type of core barrel that became the most preferred was the PQ-size, which produced larger diameter cores of better quality, than the HQ-core barrel. Once the geologist determined that limestone bedrock had been encountered, the driller removed the splitspoon sampler and drilling rods from the borehole, and prepared to rock core. The core barrel and all accompanying rods were measured prior to coring. After the core barrel and accompanying drilling rods were placed down the hole, the remaining drilling rod sticking out of the hole (called stick-up) was measured and the depth was calculated, prior to coring. Elevation was also checked in the field using an RTK differential GPS system, with associated datum and tidal corrections applied.

Figure B-43. Drill crews conducting rock coring using PQ-size, diamond impregnated core barrel.

The rock cores were taken on 5-foot intervals to at least elevation -60 feet MLLW. The driller measured the progress of the run and the pressure applied to advance the core barrel. Little to no pressure (< 100 psi) was applied, as the core barrel cut down easily under its own weight and rotational speed. Upon completion of the run the core barrel was retrieved, opened, and the core was slid onto a tray for cleaning, photographing, and logging. Once the core was logged, it was wrapped in cellophane to retain moisture, boxed, and packaged for shipment to the lab. Project information, boring id, run number, and sampling depth, top and bottom of the core, and sketch were annotated onto the inside cover and box exterior prior to storage.

5.4.1.7 Data Logging. All data collected in the field was recorded in the geologists' field notebooks. Pertinent data include, but are not limited to the depth drilled, total casing used, depth to seafloor, elevation corrections, soils and lithologies encountered, SPT blow count, missed sampling intervals, rock core run depths, recovery, and Rock Quality Designation (RQD) calculations. In addition, photographs of the core runs were made by the SAW geologist. Samples were selected for laboratory testing, manifested, and shipped to the USACE-EMU geotechnical laboratory for analysis. Sample name, depth, elevation, and type of test for each boring were then documented for record. All the documented field data was then entered into Bentley's gINT geotechnical software program, which can output detailed USACE 1836 boring logs (Attachment B-2), fence diagrams (Figures B-53 to B-71), and other products.

5.4.2. Laboratory Testing Program

The USACE Environmental & Materials Unit (EMU) geotechnical laboratory in Marietta, GA was selected to conduct the laboratory testing. A total of 103 soil samples and 104 rock samples were submitted for testing. The lab received samples early August 2013, and conducted testing until late December 2013. Soil tests included particle grain size analysis (ASTM D422), Atterburg limits (ASTM D4318), and visual classification (ASTM D2488). Rock strength tests included unconfined compressive strength (ASTM D2938), Brazilian splitting tensile strength (ASTM D3967), unit weight and specific gravity. The following is a brief description of each of the tests conducted.

5.4.2.1. Particle Grain Size Analysis. Granular soil samples were selected for laboratory grain size analysis (ASTM D422). The method is summarized in the following steps;

- A portion of the soil sample is placed into a weighing dish, usually 500 grams and is weighed (wet). NOTE: particles of cemented sand may be broken down
- The soil sample is dried and weighed again for its dry weight.
- The soil sample is placed onto a stacked series of tare weighed sieves. For granular samples, the sieves usually start at the #4 sieve, which separates gravel from sand, and runs through to the #200, which separates fines from sand. Coarser grained samples may have the addition of the 3-inch sieve to separate cobbles from the gravel fraction. Finer-grained samples may include the #230 sieve to capture very fine sand fraction. NOTE: the USCS makes no distinction between fine-grained particles that passes the #200 sieve. The sieve stack with samples is placed into a mechanical shaker box for a specified period of time.
- Upon completion of shaking, the sieve stack is broken apart, and each sieve, with soil sample fraction retained on the screen, is weighed.
- Calculations are made to determine the weight percent passing each sieve, the gradation data is graphically plotted on a logarithmic scale showing finer by weight v. grain size in millimeters.

- The relative percentages of each soil constituent (% gravel, % sand, % fines) are then assessed.

5.4.2.2. Atterburg Limits. Fine-grained soils were selected for the Atterburg limits test, which was conducted in accordance with ASTM D4318. The test is conducted to determine the liquid limit, plastic limit, and plasticity index of fine-grained soils that pass the #40 sieve. The engineering properties of silts and clay, such as shear strength and volume, will change depending upon the water content in the soil. As a very wet fine-grained soil dries, its consistency changes from a viscous liquid into a plastic state.

According EM 1110-2-1906 *"Laboratory Soil Testing"* and ASTM D4318, the liquid limit is defined as that moisture content at which the soil first shows a small change in the shear strength as the moisture is reduced. The liquid limit is determined using the liquid limit testing device. A portion of the sample is placed into the metal cup and a groove is cut down the center of the sample using a standard flat grooving tool. The cup is then repeatedly dropped 10mm at a rate of 120 blows per minute by turning the device's crank handle. The number of blows required to cause the gap to close is recorded. Several runs are made, varying the moisture content each time. The results from each run are graphed in a plot of # blows vs. moisture content. The liquid limit is the interpolated from the graphed line as the moisture content at which it takes 25 blows to cause the gapped soil to close.

The soil's plastic limit is determined by rolling out a thread of the pre-weighed, moist sample onto a flat non-porous surface, usually a glass or ceramic plate. If the soil contains significant amounts of clay, the thread will retain its shape down to a very narrow diameter. The sample is continually remolded and the test repeated. As the moisture content falls due to handling and evaporation, the thread will begin to break apart. The plastic limit is defined as the moisture at which the thread breaks apart at a diameter of 3.2 mm. The weight of the soil is measured after the test, and then upon drying 16-hours in a drying oven in order to determine its moisture content at the plastic limit (when the soil crumbled). The soil's plasticity index is determined by subtracting the plastic limit from the liquid limit.

5.4.2.3. Visual Classification. Soil samples were selected for laboratory visual classification, for the purpose of verifying and checking the geologist's soil field classification. There is little difference between field and laboratory visual classifications, except that in the field the geologist has the benefit of seeing the strata as it is sampled, with its internal soil structure/stratigraphy fairly intact, whist the laboratory has time and accompanying lab testing to facilitate his classification. The elements of the Unified Soil Classification System are; fine-grained/coarse-grained soil determination, color, moisture condition, density/consistency, hardness, gradation, and plasticity (for silts/clays).

5.4.2.4. Unconfined Compressive Strength Test. The Unconfined Compressive Strength (UCS) test is one of the most basic strength parameters for rock strength, and the most common determination performed for rock excavation. It is measured in accordance with ASTM D2938 and USACE RTH 111-89 *"Uniaxial Compressive Strength"*. The rock core test specimen, having a length to diameter ratio of 2 is placed into a loading device. The device should be capable of applying and measuring the axial load to the sample, while a chronometer or similar instrument measures the time elapsed. The specimen is then loaded uniformly and continuously until brittle failure occurs. The unconfined compressive strength is calculated by dividing the maximum load carried by the specimen during the test, by the specimen's cross-sectional area.

5.4.2.5. Splitting Tensile Strength Test. The Brazilian Splitting Tensile Strength (STS) test is another laboratory test method that is used to assess the tensile strength of the sampled rock mass. The Brazilian method (ASTM 3967 or RTH 113-80), while indirect, is far easier and practical to use than more expensive in-situ and pull-apart testing. This test method simply involves taking a disk of rock core having a length to diameter ratio of ½, and placing it on its side in the same loading apparatus used for the UCS test. The specimen is then loaded continuously until brittle failure occurs. The splitting tensile strength is calculated by dividing the product of 2 times the maximum load carried by the specimen, by the product of pi multiplied by the specimen's thickness and its diameter.

5.5 Results of Geotechnical Drilling 2013

A total of 49 geotechnical borings were drilled 8 to 20 feet into the subsurface in the Charleston Harbor Entrance Channel. The borings logs and lab are presented separately in Attachments B-2, B-3 and B-4. The strata targeted for sampling and testing lie between the present channel bottom elevation (≈ 48 feet MLLW) and the maximum proposed deepening elevation (-58 feet MLLW). Most of the borings were advanced below -60 feet MLLW. Borehole location, depth drilled, and elevation to top of rock (if encountered) is presented in Plate 11. Five borings, EC-13-B-25, -26, -29, -30, and -31 were not able to be drilled due to inclement wave and weather conditions that arose on 25AUG13, and lasted to 27AUG13. A mechanical breakdown in the Cap'n Ray's starboard spud system resulted in termination of the drilling program on 30AUG13.

Borehole information parameters critical to the project are summarized in Table B-14. These include the general location of limestone bedrock, bedrock elevation, maximum unconfined compressive strength of rock, and the general sediment types sampled.

Table B-14. Summary of USACE exploratory drilling in Charleston Harbor, August, 2013.

Boring ID	**Channel Segment**	**Channel Station**	**Seafloor Elev.**	**BOH Elev.**	**Footage Sampled**	**Predominant Sediment Type Seafloor to -58 MLLW**	**Elevation Top of Rock**	**UCS Max**	**Unit Interpretation**
EC-13-B-1	2	852+54	-51.8	-61	9.2	Silt	---	---	Cooper Marl Fm
EC-13-B-2	2	827+48	-52.6	-60.4	7.8	Silt	---	---	Cooper Marl Fm
EC-13-B-3	2	824+34	-56.4	-63.3	6.9	Silt	---	---	Cooper Marl Fm
EC-13-B-4	3	788+30	-53.2	-72.2	19	Silty Sand	---	---	Cooper Marl Fm
EC-13-B-5	3	750+00	-44.1	-60.4	16.3	Organic Silt & Silty Sand	---	---	Cooper Marl Fm
EC-13-B-6	4	738+79	-43.2	-61.8	18.6	Organic Silt & Silty Sand	---	---	Cooper Marl Fm
EC-13-B-7	4	729+68	-44.2	-65	20.8	Organic Silt & Silty Sand	---	---	Cooper Marl Fm
EC-13-B-8	4	717+24	-43.9	-63.2	19.3	Organic Silt & Silt	---	---	Cooper Marl Fm
EC-13-B-9	4	701+46	-43.2	-64.9	21.7	Organic Silt & Silt	---	---	Cooper Marl

Boring ID	Channel Segment	Channel Station	Seafloor Elev.	BOH Elev.	Footage Sampled	Predominant Sediment Type Seafloor to -58 MLLW	Elevation Top of Rock	UCS Max	Unit Interpretation
									Fm
EC-13-B-10	4	694+50	-45	-65.1	20.1	Organic Clay & Silty Clay	---	---	Cooper Marl Fm
EC-13-B-11	5	687+49	-45.2	-61.8	16.6	Organic Silt & Sandy Gravel	---	---	Edge of Edisto Fm?
EC-13-B-12	5	684+10	-46	-62.8	16.8	Organic Silt & Gravelly Sand	---	---	Edge of Edisto Fm?
EC-13-B-13	5	675+47	-44.2	-65.2	21	Organic Clay & Silty Sand	---	---	Edge of Edisto Fm?
EC-13-B-14	5	668+00	-44.5	-64.4	19.9	Organic Silt & Sand	---	---	Edge of Edisto Fm?
EC-13-B-15	5	665+15	-46	-64.1	18.1	Organic Silt & Sand	---	---	Edge of Edisto Fm?
EC-13-B-16	6	630+17	-43.9	-61.4	17.5	Organic Silt & Gravel	---	---	Edge of Edisto Fm?
EC-13-B-17	6	620+22	-46.7	-62.3	15.6	Organic Silt & Silty Sand	---	---	Edge of Edisto Fm?
EC-13-B-18	6	616+45	-46.6	-63.4	16.8	Organic Silt & Limestone	-51.9	140 psi	Edisto Fm
EC-13-B-19	6	605+95	-47.7	-60.1	12.4	Organic Silt & Sand	---	---	Edge of Edisto Fm?
EC-13-B-20	6	601+75	-50.3	-62.2	11.9	Limestone	-50.7	210 psi	Edisto Fm
EC-13-B-21	7	578+27	-48	-62	14	Limestone	-51.4	158 psi	Edisto Fm
EC-13-B-22	7	553+37	-47.6	-60.7	13.1	Organic Silt & Sand	---	---	Edge of Edisto Fm?
EC-13-B-23	7	556+50	-51.4	-61.4	10	Organic Silt & Silt	---	---	Cooper Marl Fm
EC-13-B-24	7	538+71	-52.7	-60.9	8.2	Gravel & Limestone	-52.9	80 psi	Edisto Fm
EC-13-B-27	8	509+02	-51	-60.2	9.2	Sand	---	---	Edge of Edisto Fm?
EC-13-B-28	8	493+18	-51.1	-62.6	11.5	Limestone	-51.1	98 psi	Edisto Fm
EC-13-B-32	10	422+64	-53.7	-60.2	6.5	Limestone	-53.7	189 psi	Edisto Fm
EC-13-B-33	10	416+55	-50.7	-62.7	12	Limestone	-52.5	351 psi	Edisto Fm
EC-13-B-34	10	396+69	-52.9	-60.9	8	Limestone	-53.9	125 psi	Edisto Fm
EC-13-B-36	10	385+54	-49.1	-57.6	8.5	Limestone	-52.6	184 psi	Edisto Fm
EC-13-B-35	10	382+47	-52	-63.7	11.7	Limestone	-52	195 psi	Edisto Fm
EC-13-B-37	10	373+09	-51.4	-61	9.6	Limestone	-53.1	175 psi	Edisto Fm
EC-13-B-38	11	362+86	-53.7	-60.9	7.2	Limestone	-54.4	33 psi	Edisto Fm
EC-13-B-39	11	353+60	-52.2	-69.6	17.4	Limestone	-52.2	249 psi	Edisto Fm
EC-13-B-40	11	349+29	-52.2	-63.8	11.6	Limestone	-52.2	295 psi	Edisto Fm
EC-13-B-41	11	334+51	-51.6	-60.6	9	Limestone	-51.6	226 psi	Edisto Fm
EC-13-B-42	11	333+95	-50.8	-62.8	12	Limestone	-50.8	223 psi	Edisto Fm
EC-13-B-43	11	323+13	-51.8	-63.2	11.4	Limestone	-51.8	416 psi	Edisto Fm
EC-13-B-45	12	309+98	-51.3	-62.8	11.5	Limestone	-51.3	227 psi	Edisto Fm
EC-13-B-44	12	309+65	-53.2	-61.8	8.6	Limestone	-53.2	114 psi	Edisto Fm
EC-13-B-46	12	298+55	-53.6	-62	8.4	Limestone	-53.6	138 psi	Edisto Fm
EC-13-B-47	12	294+19	-53.1	-61.1	8	Limestone	-53.1	130 psi	Edisto Fm
EC-13-B-48	12	290+60	-49.7	-62.8	13.1	Limestone & Gravel & Sand	-49.7	209 psi	Edisto Fm
EC-13-B-49	12	281+24	-49.6	-61.3	11.7	Limestone	-49.6	88 psi	Edisto Fm

Boring ID	Channel Segment	Channel Station	Seafloor Elev.	BOH Elev.	Footage Sampled	Predominant Sediment Type Seafloor to -58 MLLW	Elevation Top of Rock	UCS Max	Unit Interpretation
EC-13-B-50	13	260+35	-49.4	-64.8	15.4	Limestone	-49.4	115 psi	Edisto Fm
EC-13-B-51	13	250+18	-49.5	-61	11.5	Limestone	-49.5	95 psi	Edisto Fm
EC-13-B-52	13	243+68	-49.7	-60.8	11.1	Limestone	-49.7	107 psi	Edisto Fm
EC-13-B-53	13	230+90	-50	-60.7	10.7	Sand	---	---	Modern?
EC-13-B-54	14	211+63	-50	-60.9	10.9	Sand	---	---	Modern?
EC-13-B-55	14	177+10	-50.3	-65.3	15	Sand, Silt & Gravel	---	---	Modern?

5.6 Subsurface Fence Diagrams

Drilling data from the 2013 study were consolidated with historical data using gINT geotechnical software in order to delineate the subsurface conditions within the entrance channel. Fence diagrams for the north (left) and south (right) sides of the channel were drafted for 19 of 21 channel subsections. The outermost channel segments, EC-20 and EC-21, were not delineated because the existing bathymetry and washprobe refusal data indicates there is no rock present within the proposed dredging prism. Color coded bathymetric data from 25JUN13 is provided and the average depth of the channel along profile is drawn on each fence profile. The maximum proposed dredge depth is also shown. Material lying between these two lines is considered in-situ will likely be encountered during deepening. Washprobes are denoted by elevation that indicates a refusal depth. Generally, medium to hard silts and clays, and dark silty sands are associated with the Cooper Formation. Limestone, shelly gravels, coquina, and dense gray shelly to silty cemented sands are associated with the Edisto Formation. Dense, poorly graded quartz sand that lies above the Edisto Formation is interpreted to belong to the Marks Head Formation, based upon the work of Weems and Lemon (1993). Very soft clays and deep refusal depths are interpreted to represent buried paleo-fluvial channels. Top of rock was delineated where the Edisto Formation is inferred to lie in the subsurface.

5.7 Entrance Channel Stratigraphy

5.7.1. Entrance Channel, Segment EC-1

A total of 10 borings and 2 washprobes were used to describe the subsurface conditions within segment EC-1 in cross-sectional profile, as shown in Figure B-51. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth within this segment ranges in depth from -48 feet MLLW along the channel banks to a maximum depth of -56 feet MLLW between stations 865+00 and 860+00. The average channel depth along the northern fence profile is -40 feet MLLW, while the southern fence profile is deeper at -51.5 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Cooper Formation underlies all of channel segment EC-1 and is the predominant lithologic unit based upon borings EC-69-89, EC-73-89, CHDVC50-1-1-86, CHDVC-52-1-1-86, and EC-72-89 which penetrate to -55 to -64 feet MLLW. Within the dredging prism, the Cooper Formation consists of lean inelastic silt which grades laterally into elastic silt and silty-clayey sand, with some interbedded lean clay. SPT N-values from historical borings EC-69-89, EC-71-89, EC-73-89, EC-70-89, and EC-72-89 indicate that the fine-grained materials range from soft to very stiff, while the granular materials range from loose to medium dense. Available subsurface data indicates that there is no rock present within the dredging prism of EC-1.

5.7.2. Entrance Channel, Segment EC-2

A total of 14 borings and 4 washprobes were used to describe the subsurface conditions within segment EC-2 in cross-sectional profile, as shown in Figure B-52. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth within this segment ranges in depth from -48 to -56 feet MLLW. The channel banks appear to be steeper than in EC-1 and range in depth from -52 to -54 feet MLLW. Channel segment EC-2 reaches a maximum depth of -58 feet MLLW between stations 835+00 and 810+00. The average channel depth along the northern fence profile is -51.5 feet MLLW, while the southern fence profile is slightly deeper at -52.0 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Cooper Formation is the predominant lithologic unit within EC-2 based upon the material descriptions in borings EC-13-B-1, EC-13-B-3, EC-156-98, EC-77-90, EC-137-90, EC-13-B-2, and EC-76-89, which penetrate to -55 to -63 feet MLLW. The presence of limestone gravel, cemented shelly sands and coquina in borings EC-77-90, CHDVC-55-1-1-86, EC-76-89, and EC-78-90 suggests that the Edisto Formation once overlaid the Cooper Formation in this channel segment, possibly as an erosional outlier. This material was then removed during the last harbor deepening which exposed the underlying Cooper Formation. Within the proposed dredging prism, the Cooper Formation consists of lean inelastic silt which grades laterally into silty-clayey sand. SPT N-values from borings EC-13-B-1, EC-13-B-2, EC-13-B-3, EC-156-98, EC-77-90, EC-76-89 and EC-78-90 indicates that the fine-grained material ranges from soft to very stiff, while the granular material ranges from loose to dense. Available subsurface data indicates that the limestone may have once been present at -37 feet MLLW from station 827+00 seaward; however, this material has been removed. Small lenses of very dense clayey sand are present along the southern side of the channel between stations 847+00 and 842+00, but this is considered limited in extent. Available subsurface data indicate that there is no rock present within the dredging prism of EC-2.

5.7.3. Entrance Channel, Segment EC-3

A total of 15 borings and 6 washprobes were used to describe the subsurface conditions within segment EC-3 in cross-sectional profile, as shown in Figure B-53. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth within this segment ranges in depth from -46 to -56 feet MLLW. The southern channel bank is much broader and less steep than the northern bank. Channel segment EC-3 reaches its maximum depth between stations 790+00 and 760+00. The average channel depth along both northern and southern fence profiles is -48.0 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Cooper Formation is the predominant lithologic unit within EC-3 based upon the material descriptions in borings EC-46-88, EC-13-B-4, EC-80-90, CHDVC-58-1-1-86, CHDVC-57-1-1-86, EC-48-88, EC-13-B-5, EC-78-90, EC-47-88, EC-136-90, and CHDVC-60-1-1-86, which penetrate to -50 to -57 feet MLLW. The presence of limestone, limestone gravel, cemented shelly sands and coquina in borings EC-80-90, EC-82A-90, EC-84A-90, EC-78-90, CHDVC-56-1-1-86, and CHDVC-60-1-1-86 suggests that the Edisto Formation once overlaid the Cooper Formation in this channel segment, possibly as an erosional outlier. This material was then removed during the last harbor deepening which exposed the underlying Cooper Formation. Presently, the Edisto Formation is only present along the south bank of the channel between stations 790+00 and 780+00. Throughout the remainder of the dredge prism, the Cooper Formation consists of lean inelastic silt with 1-4 foot thick lenses of fat clay, which grades laterally into silty-clayey sand. SPT N-values from borings EC-13-B-4, EC-80-90, EC-13-B-5, EC-78-90, EC-47-88, EC-49-88 and EC-136-90 indicates that the fine-grained material ranges from medium-stiff to stiff, while the granular material ranges from loose to dense. Available subsurface data indicates that the limestone may have once been present along the north side of the channel between stations 780 + 00 and 745+00, and along the southern side of the channel from station 793+80 to 775+00. Present bathymetric surveys indicate that this material has since been removed by dredging. Much of EC-3 is free of rock, with exception to the south bank between stations 793+80 and 780+00.

5.7.4. Entrance Channel, Segment EC-4

A total of 15 borings and 9 washprobes were used to describe the subsurface conditions within segment EC-4 in cross-sectional profile, as shown in Figure B-54. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth within this segment is ranges in depth from -40 to -52 feet MLLW. Both channel banks appear to be uniform in slope and there are no large bathymetric features such as depressions or shoals present. The average channel depth along both northern and southern fence profiles is -44.0 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Cooper Formation is the predominant lithologic unit within EC-4 based upon the material descriptions in borings EC-13-B-6, EC-13-B-7, EC-133-90, EC-13-B-10, CHDVC-60-1-1-86, CHDVC-62-1-186, EC-13-B-8, EC-132-90, and EC-13-B-11, which penetrate to a maximum of -65 feet MLLW. Coquina, limestone gravel, and calcareous cemented sand described in CHDVC-63-1-1-86, EC-133-90, CHDVC-60-1-1-86, EC-51-88, CHDVC-62-1-1-86, and EC-132-90 suggests that the Edisto Formation once overlain the Cooper Formation in this channel segment, prior to the last dredge deepening. This material was subsequently removed, exposing the underlying Cooper Formation. The Cooper Formation forms the underlying foundation strata throughout EC-4. The strata consist predominantly of lean inelastic silt with significant amounts of silty sand present from station 738+00 to 715+00. SPT N-values from within the Cooper Formation indicate that the fine-grained material ranges from medium-stiff to stiff, while the granular material tends to be loose. The Edisto Formation is present as a thin layer weakly cemented shelly to calcareous sands, gravels and coquina that extends from station 725+00 to station 700+00 on the north side of the channel, and from station 735+00 to 690+00 on the south side of the channel.

5.7.5. Entrance Channel, Segment EC-5

A total of 14 borings and 10 washprobes were used to describe the subsurface conditions within segment EC-5 in cross-sectional profile, as shown in Figure B-55. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -40 to -52 feet MLLW. As in EC-4, both channel banks appear to be uniform in slope and there are no large bathymetric features such as depressions or shoals present. The average depth along the northern fence profile is -44.0 feet MLLW, while the southern profile is slightly deeper at -45 MLLW. The

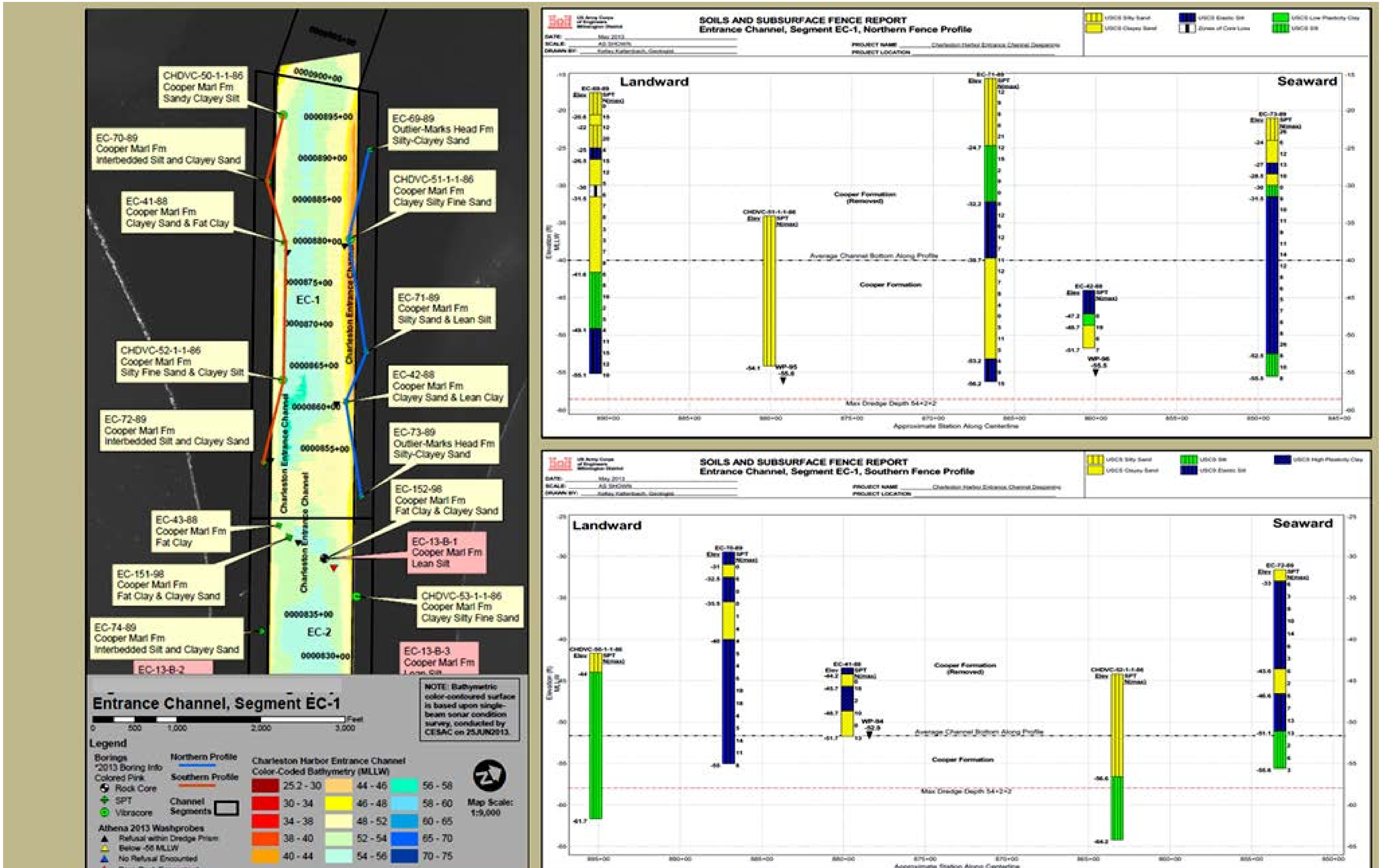


Figure B-44. Fence Diagram of Entrance Channel, Segment EC-1

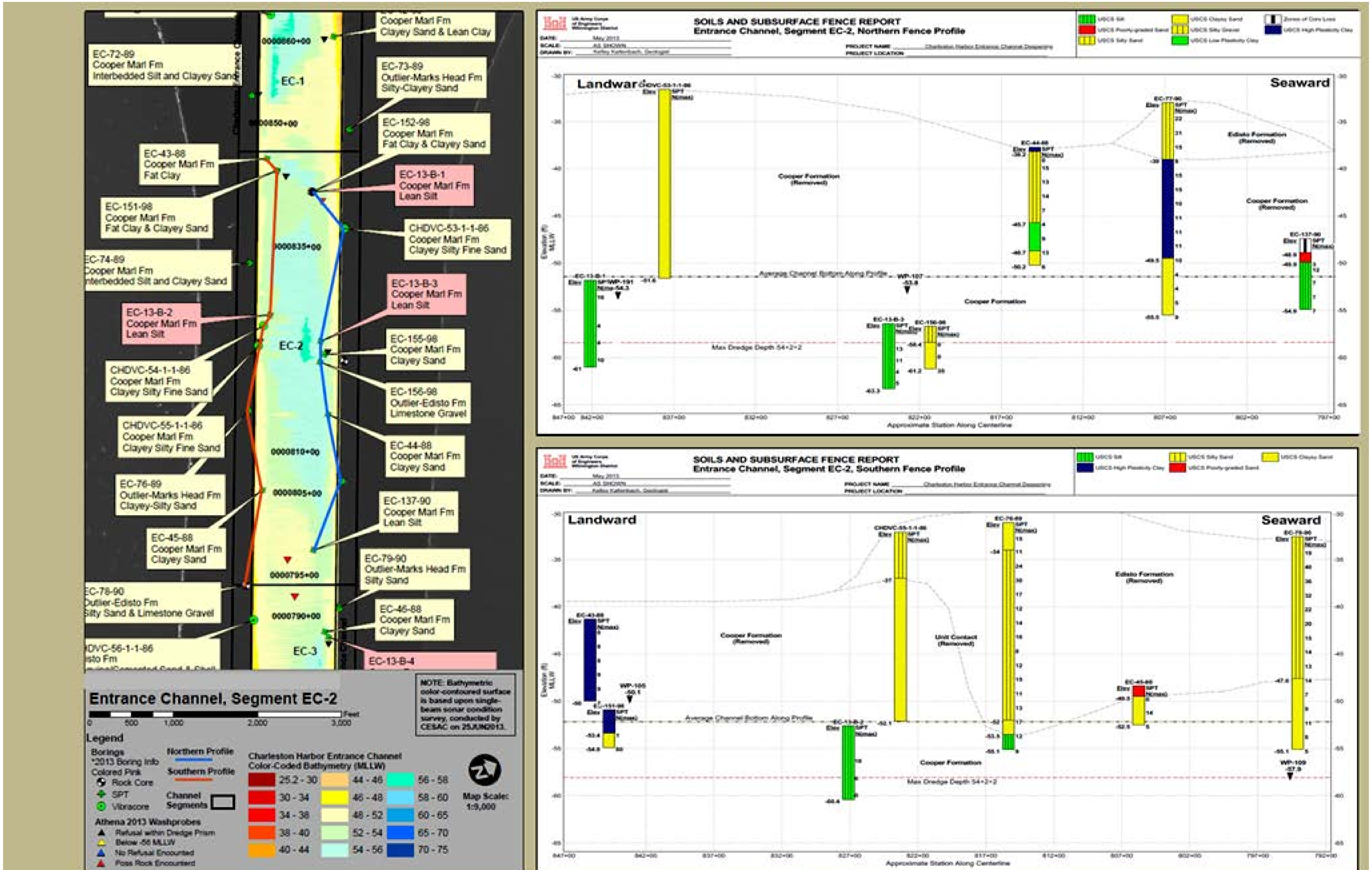


Figure B-45. Fence Diagram of Entrance Channel, Segment EC-2

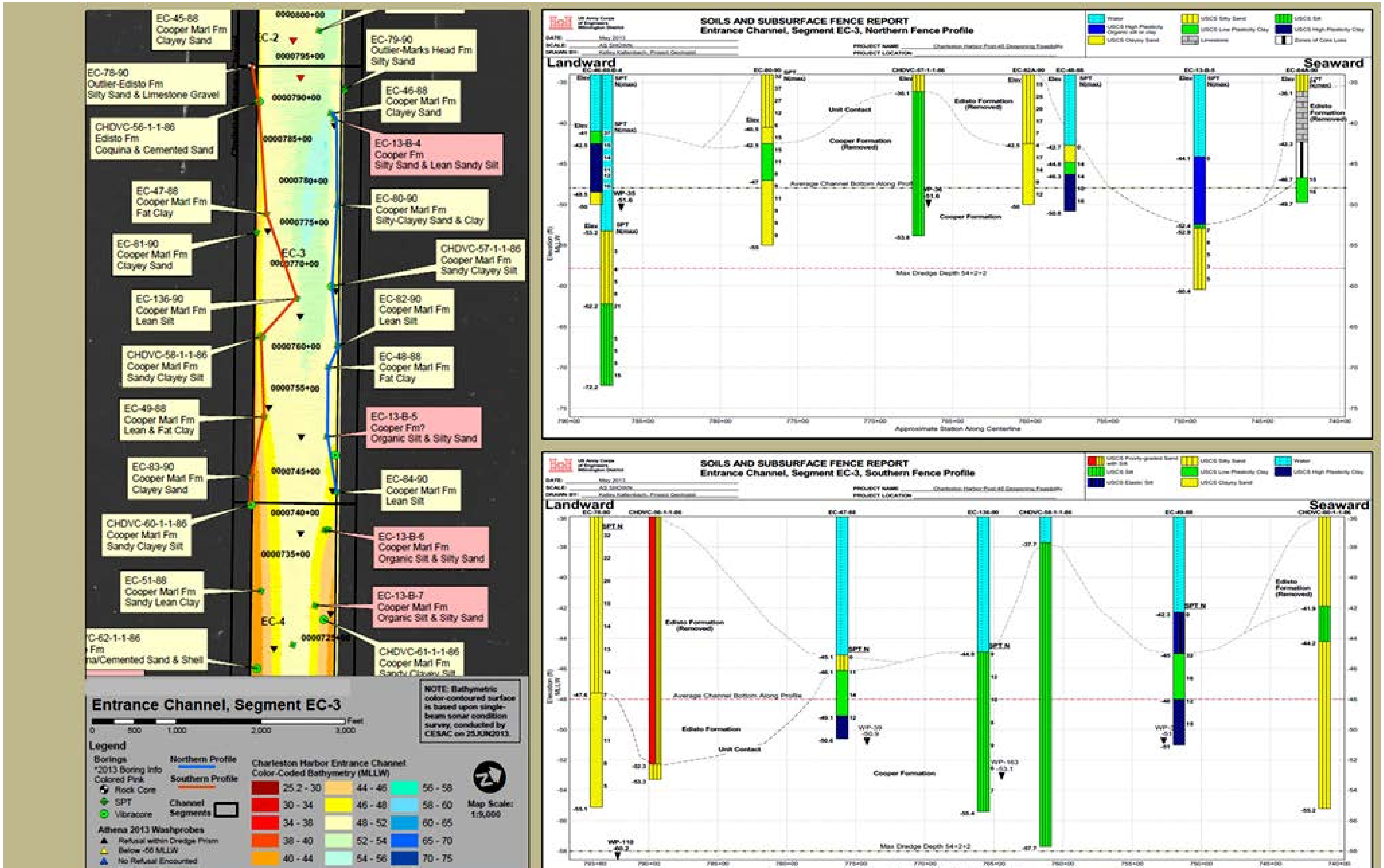


Figure B-46. Fence Diagram of Entrance Channel, Segment EC-3

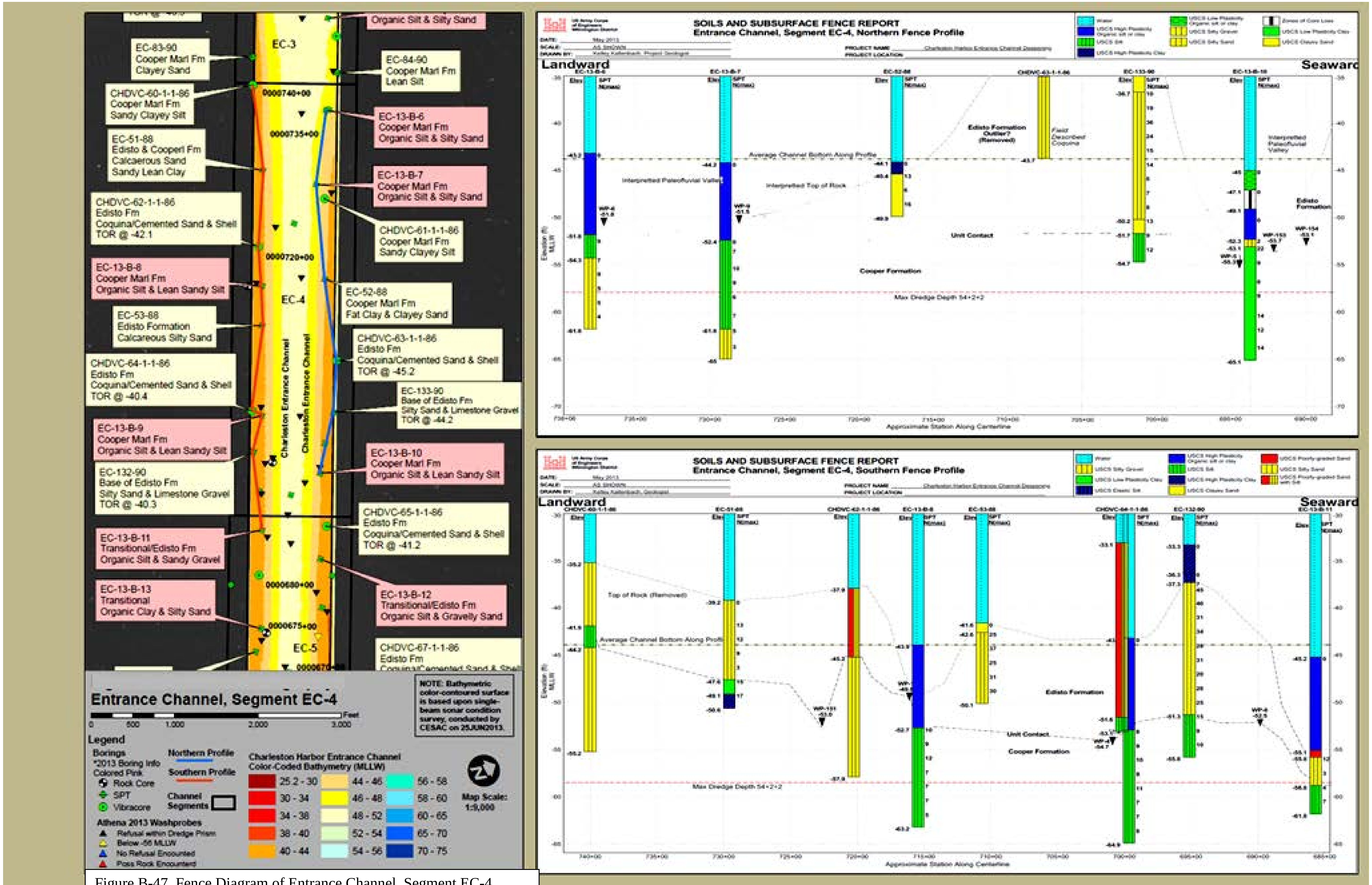


Figure B-47. Fence Diagram of Entrance Channel, Segment EC-4

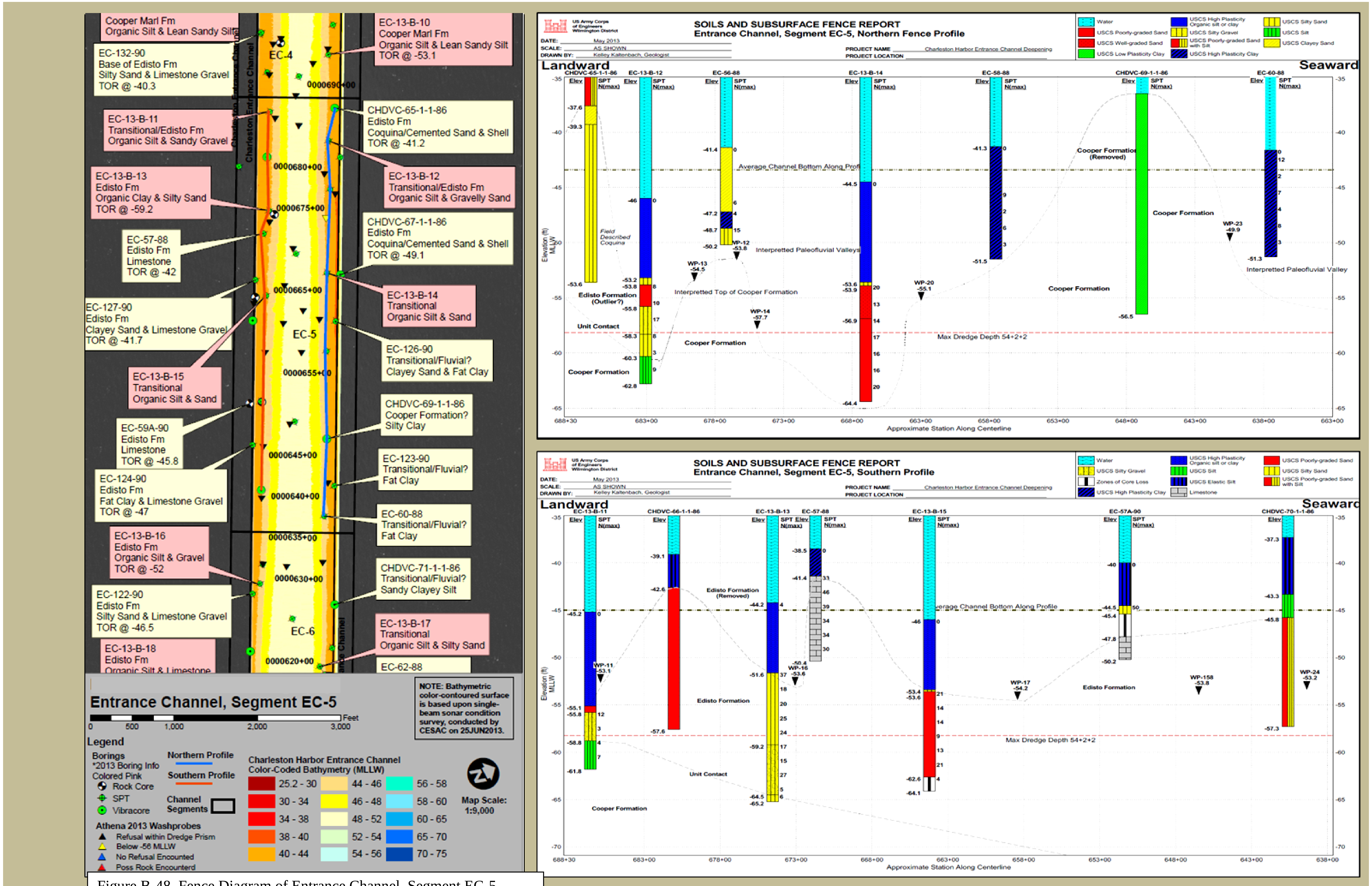


Figure B-48. Fence Diagram of Entrance Channel, Segment EC-5

maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along depth are not shown. The Edisto Formation is the predominant lithologic unit along the southern side of EC-5 and it overlies the Cooper Formation based upon borings CHDVC-65-1-1-86, EC-13-B-12, CHDVC-69-1-1-86, EC-13-B-11, EC-57-88, EC-57A-90 and CHDVC-66-1-1-86. The Cooper Formation appears to plunge into the subsurface to the south and east seaward of station 673+00; however the unit appears to form a ridge (shown in boring CHDVC-69-1-1-86) between stations 653+00 and 643+00 on the north side of the channel. Here, it is bounded by what is interpreted to be two clay-filled paleofluvial valleys interpreted from borings EC-58-88 and EC-60-88. Borings CHDVC-66-1-1-86, EC-13-B-13, EC-57-88, EC-13-B-15, EC-57A-90, and CHDVC-70-1-1-86 contain varying amounts of cemented, dense calcareous sands and gravels, coquina, and limestone which is more prevalent along the southern side of the channel than the north. Several north-south buried paleofluvial valleys appear to be incised into the Edisto and Cooper Formations. These interpreted paleofluvial valleys are in-filled by very soft fat clay. SPT N-values from borings drilled into the Edisto Formation indicate that the granular material ranges from medium dense to very dense. The available subsurface data suggests that the top of limestone bedrock rock will be encountered within the proposed dredging prism along the southern side of the channel, between station 683+00 and station 638+00.

5.7.6. Entrance Channel, Segment EC-6

A total of 14 borings and 9 washprobes were used to describe the subsurface conditions within segment EC-6 in cross-sectional profile, as shown in Figure B-56. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -40 to -54 feet MLLW. Both channel banks are uniform in slope and character, while the channel centerline varies in depth from -48 to -52 feet MLLW. The average depth along the northern fence profile is -44.0 feet MLLW, while the southern profile is slightly deeper at -46 MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-6 based upon borings EC-13-B-17, CHDVC-73-1-1-86, EC-13-B-19, CHDVC-69-1-1-86, EC-117-90, EC-57-88, CHDVC-75-1-1-86, EC-61-88, EC-13-B-16, CHDVC-72-1-1, EC-13-B-18, EC-63-88, and EC-13-B-20. Of these borings, EC-13-B-18 and EC-13-B-20 were rock cores that sampled intact limestone. The limestone appears to be more predominant along the southern side of the channel than in the north. The Edisto Formation along the northern side of the channel is better characterized as a weakly cemented, calcareous shelly-silty sand/gravel than a limestone. This may be due to differences in cementation, facies changes within unit, or field classification differences among the many workers that have drilled and sampled this stratum. SPT N-values from borings drilled into the Edisto Formation indicate that these granular materials range from medium dense to dense. The available subsurface data suggests that the top of limestone bedrock will be encountered within the proposed dredging prism between stations 631+00 and 580+00, at depths ranging from -58 to -48 feet MLLW.

5.7.7. Entrance Channel, Segment EC-7

A total of 14 borings and 9 washprobes were used to describe the subsurface conditions within segment EC-7 in cross-sectional profile, as shown in Figure B-57. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -46 to -54 feet MLLW. Both channel banks are uniform in slope and character, while the channel centerline varies in depth from -48 to -52 feet MLLW. The average depth along the northern fence profile is -45.0 feet

MLLW, while the southern profile is deeper at -48 MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-7 based upon borings CHDVC-75-1-1-86, EC-13-B-22, EC-112A-90, CHDVC-77-1-1-86, EC-111-90, EC-33-88, EC-13-B-21, EC-115-90, CHDVC-76-1-1, EC-22-88, EC-63-88, EC-13-B-24 and CHDVC-78-1-1-86, which penetrate to a maximum depth of -62 feet MLLW. Of these borings, EC-122A-90, EC-13-B-21 and EC-13-B-24 were rock cores that sampled intact limestone. The Cooper Formation was encountered at a relatively shallow depth (-54.9 MLLW) within boring EC-13-B-23, however its occurrence is considered limited. Within the proposed dredging prism the Edisto Formation is characterized as a fossiliferous limestone, coquina, calcareous shelly to silty sand and/or gravel. The differences in characterization depend upon natural differences in cementation, and classification differences among the many workers that have drilled and sampled this stratum. SPT N-values from borings drilled within the dredging prism indicate that these granular materials are generally medium dense. Available subsurface data suggests that the top of limestone bedrock surface will be encountered within the proposed dredging prism between stations 585+00 and 525+00, at depths ranging from -58 to -45 feet MLLW.

5.7.8. Entrance Channel, Segment EC-8

A total of 14 borings and 7 washprobes were used to describe the subsurface conditions within segment EC-8 in cross-sectional profile, as shown in Figure B-58. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -46 to -54 feet MLLW. The average depth along both northern and southern fence profiles is -48.0 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-8 based upon borings EC-23-88, EC-109-90, EC-66-89, EC-105-90, CHDVC-81-1-1-86, CHDVC-78-1-1-86, EC-110-90, EC-24-88A, EC-108-90, and EC-13-B-28, which penetrate to a maximum depth of -62 feet MLLW. Of these borings, EC-24-88A and EC-13-B-28 are rock cores that sampled intact limestone. The remainder of the borings was advanced by SPT or vibracore, which usually broke the limestone bedrock down into disarticulated material that was historically described as limestone rock fragments, cemented sand, gravel, or shelly sand with gravel fragments. SPT N-values from borings drilled into the Edisto Formation indicate that this granular material is generally medium dense. Available subsurface data suggests that there may be a large buried paleofluvial valley that transects EC-8 between stations 510+00 and 490+00 on the northern side, and stations 525+00 to 509+00 on the southern side. Limestone bedrock is believed to exist on either side of this channel, and the top of bedrock surface is considered to coincide with the existing bathymetric surface.

5.7.9. Entrance Channel, Segment EC-9

A total of 12 borings and 9 washprobes were used to describe the subsurface conditions within segment EC-9 in cross-sectional profile, as shown in Figure B-59. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -46 to -54 feet MLLW. The average depth along the northern fence profile is -48.0 feet MLLW, while the average depth

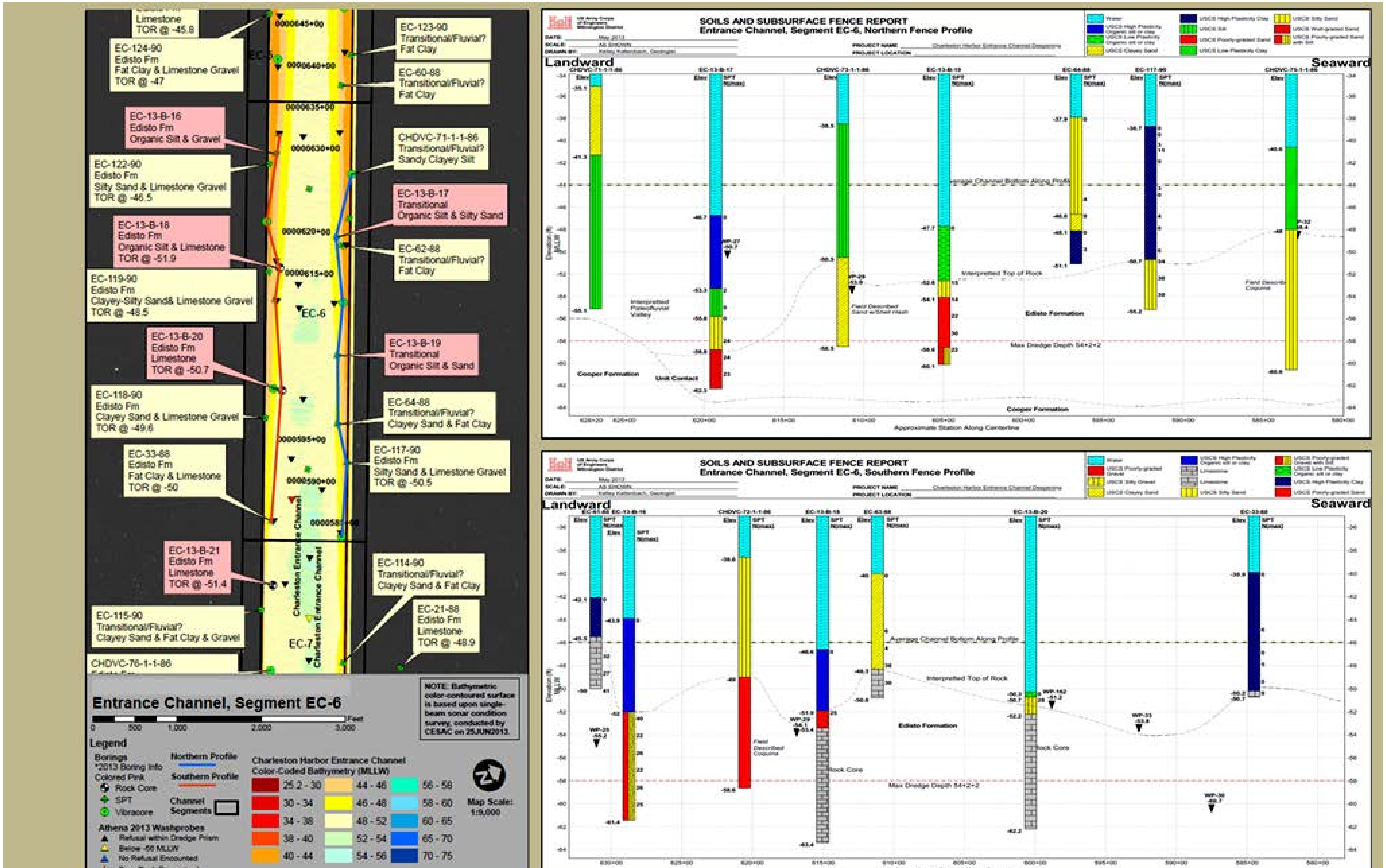


Figure B-49. Fence Diagram of Entrance Channel, Segment EC-6

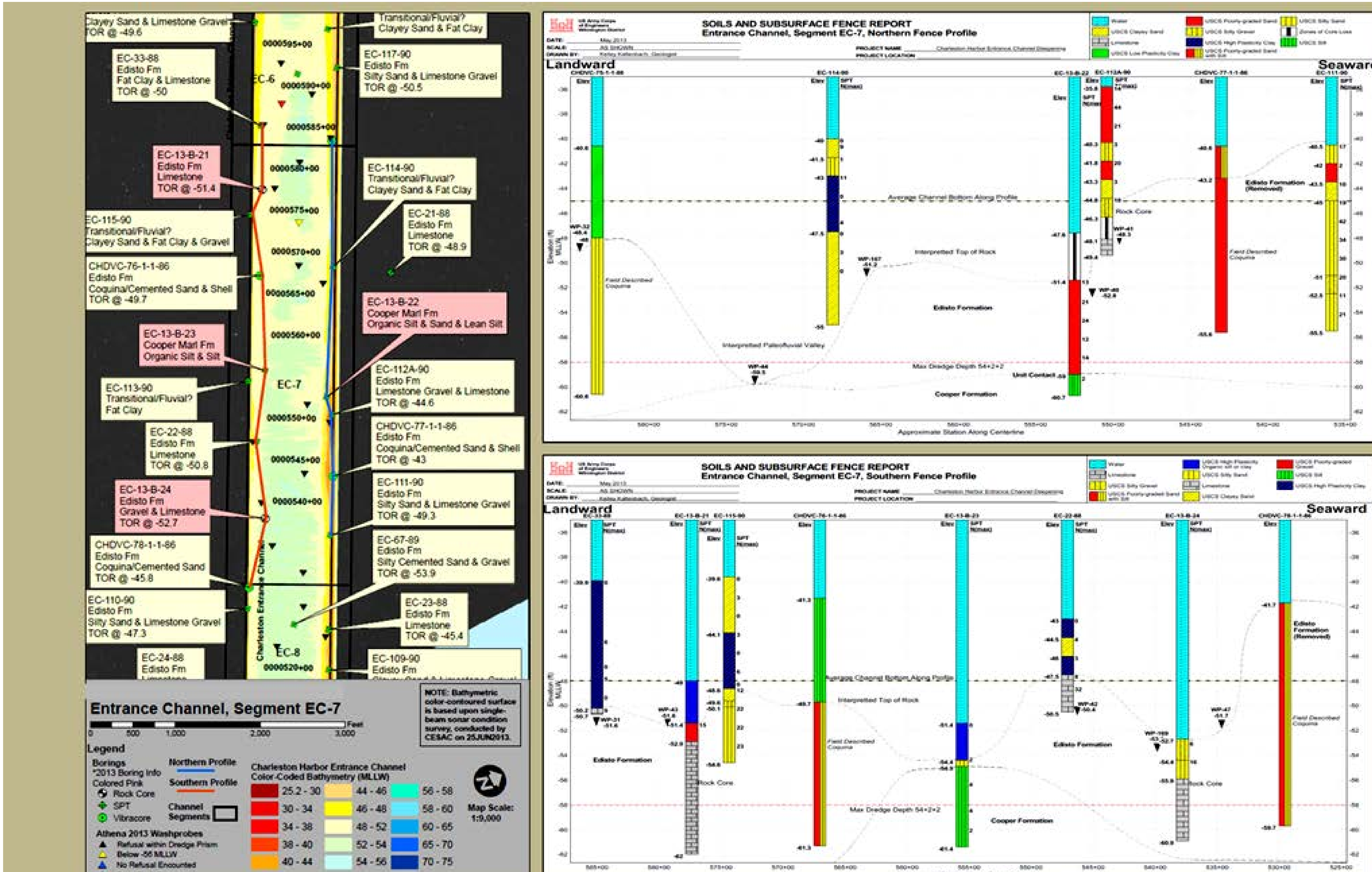


Figure B-50. Fence Diagram of Entrance Channel, Segment EC-7

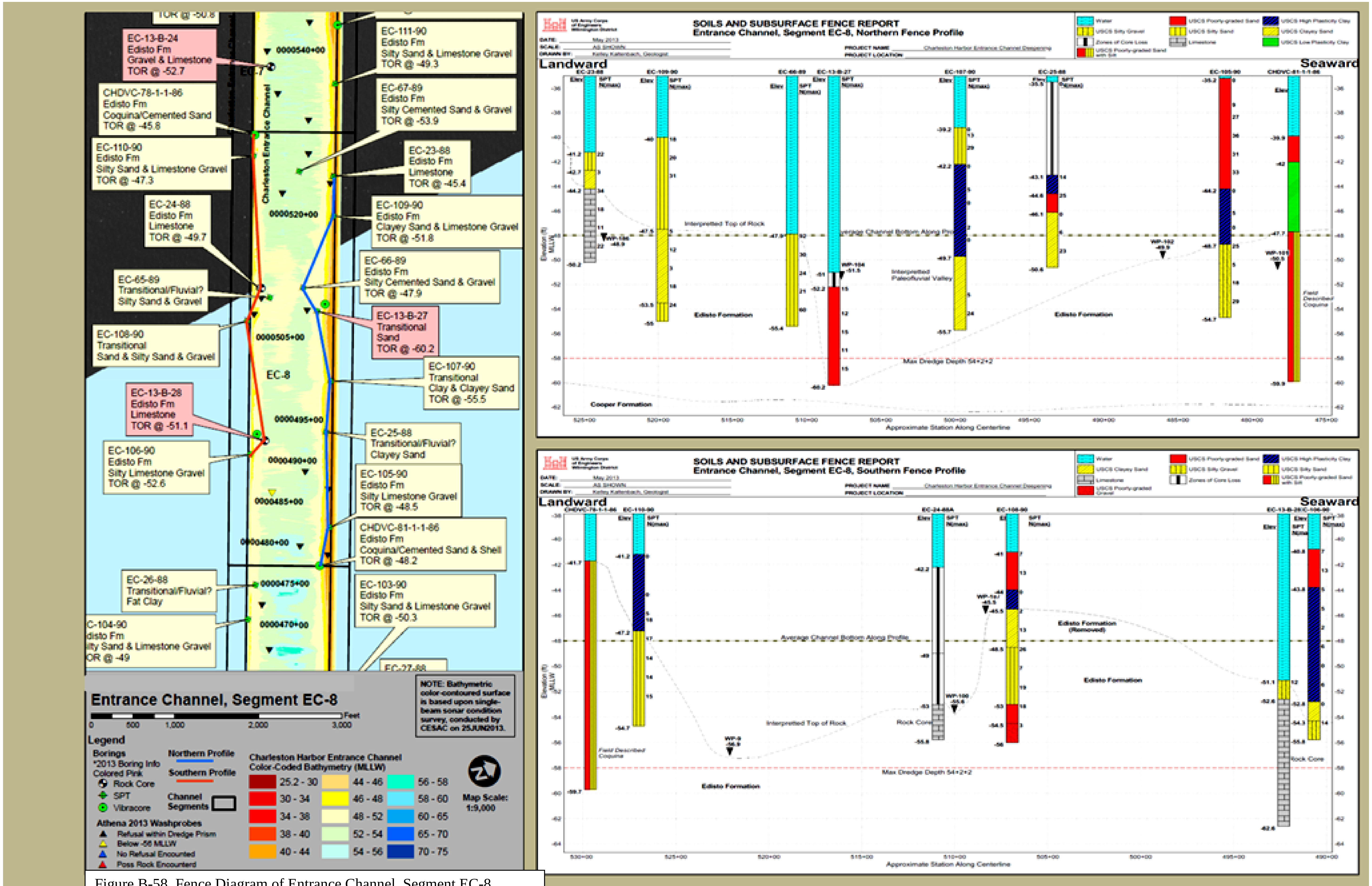


Figure B-58. Fence Diagram of Entrance Channel, Segment EC-8

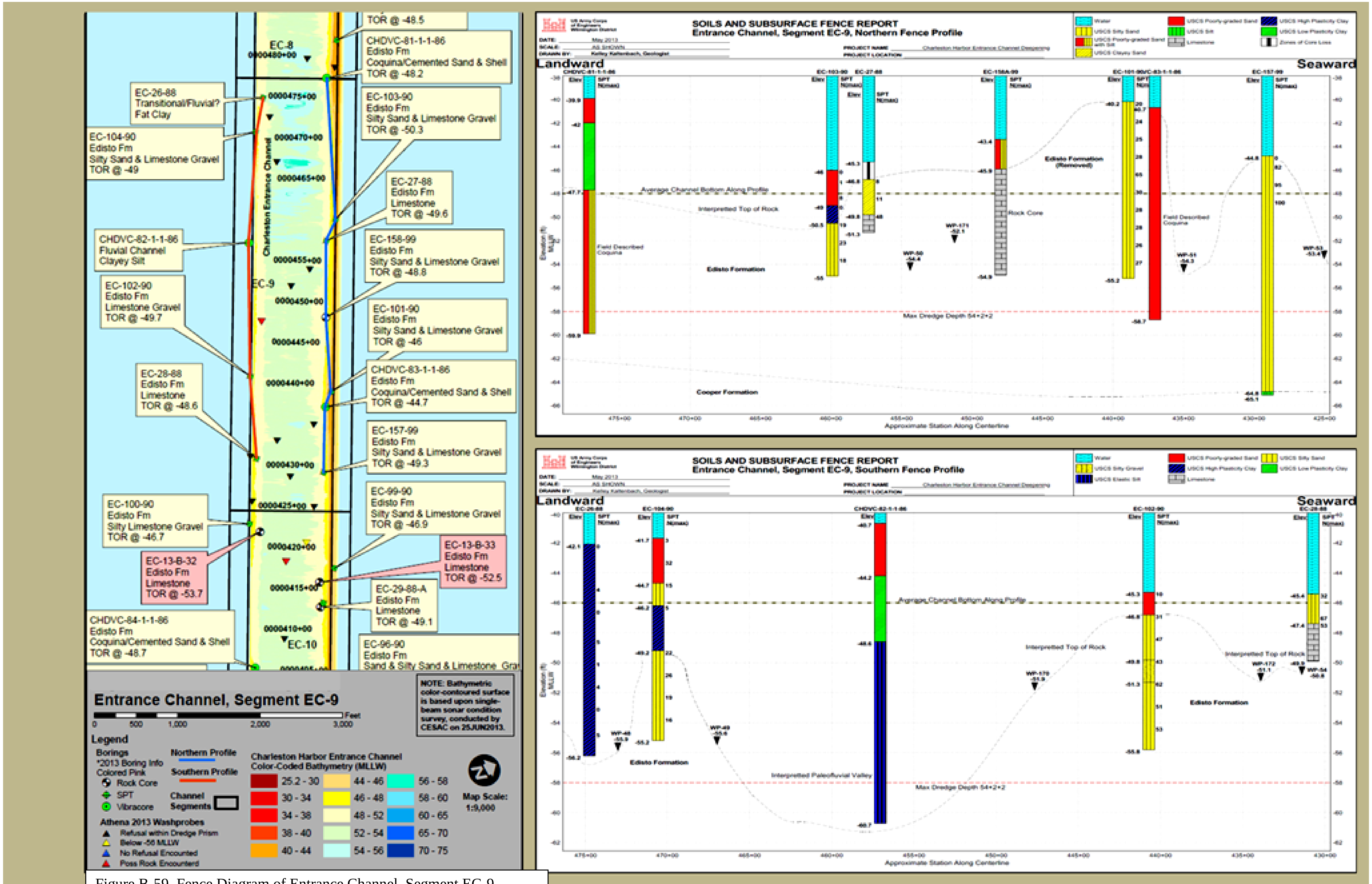


Figure B-59. Fence Diagram of Entrance Channel, Segment EC-9

along the southern profile is -46 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-9 based upon the description of materials in borings CHDVC-81-1-1-86, EC-103-90, EC-27-88, EC-158A-99, EC-101-90, CHDVC-83-1-1-86, EC-104-90, EC-102-90 and EC-28-88 which penetrate the dredge prism to a maximum depth of -64 feet MLLW. Of these borings, only EC-158A-99 is a rock core that sampled intact limestone. The remainder of the borings was advanced by SPT or vibracore. Within the proposed dredging prism, the Edisto Formation has been characterized as coquina, silty calcareous sand, cemented sand with limestone gravel, limestone gravel, or limestone. SPT N-values from borings drilled into this unit indicate that the granular material within the dredging prism is generally medium dense to very dense. Boring data from CHDVC-82-1-1-86 suggests that there may be a buried paleofluvial valley between stations 470+00 and 445+00 on the south side of EC-9. There are no similar features found along the northern profile. The available subsurface data indicates that limestone bedrock will be encountered within the proposed dredging prism for much of the length of segment EC-9. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface. The exception to this would be the subsurface vicinity of the paleofluvial channel located between stations 470+00 and 445+00, where the top of rock surface is projected below the existing average bathymetric surface.

5.7.10. Entrance Channel, Segment EC-10

A total of 17 borings and 1 washprobe were used to describe the subsurface conditions within segment EC-10 in cross-sectional profile, as shown in Figure B-60. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -46 to -54 feet MLLW. The average depth along the northern fence profile is -44.0 feet MLLW, while the average depth along the southern profile is -50 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-10 based upon the description of materials in all of the borings drilled within EC-10 (Figure B-60). Intact limestone rock cores were recovered from borings EC-13-B-33, EC-29-88A, EC-13-B-36, EC-13-B-37, EC-13-B-32, EC-13-B-34 and EC-13-B-35. The Edisto Formation may extend to depths greater than -64.0 feet based upon existing drilling logs. The remaining borings that were advanced by SPT or vibracore characterize the unit as consisting of coquina, silty calcareous sand, cemented sand with limestone gravel, or as sand with gravel. SPT N-values indicate that the material within the dredging prism are generally medium dense to very dense. The available subsurface data indicates that limestone bedrock will be encountered within much of the proposed dredging prism from station 425+00 to station 370+00. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface.

5.7.11. Entrance Channel, Segment EC-11

A total of 14 borings and 8 washprobes were used to describe the subsurface conditions within segment EC-11 in cross-sectional profile, as shown in Figure B-61. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -46 to -54 feet MLLW. The average depth along both northern and southern fence profiles is -48 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-11 based upon the description of materials in all of the borings drilled within the channel segment. Intact limestone rock cores were recovered from borings EC-13-B-39, EC-13-B-41, EC-13-B-43, EC-13-B-38, EC-13-B-40, EC-87-89 and EC-13-B-42. The Edisto Formation may extend to depths greater than -69.0 feet based upon the existing drilling logs. The remaining borings that were advanced by SPT or vibracore characterize the unit as consisting of coquina, silty calcareous sand, and cemented sand with limestone gravel. SPT N-values indicate that the limestone is generally soft and weakly cemented, and that the material within the dredging prism are generally medium dense to very dense. The available subsurface data indicates that limestone bedrock will be encountered throughout much of the proposed dredging prism from station 370+00 to station 320+00. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface. Potential exception to this is the presence of two small valley or trough features that are located between stations 330+00 and 325+00 along the northern side of the channel, and between stations 355+00 and 345+00 on the southern side. The degree to which these features are in-filled with unconsolidated sediment (if at all) is unknown.

5.7.12. Entrance Channel, Segment EC-12

A total of 11 borings and 1 washprobe were used to describe the subsurface conditions within segment EC-12 in cross-sectional profile, as shown in Figure B-62. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -54 feet MLLW. The average depth along the northern fence profile is -48 feet MLLW, while the southern fence profile is deeper at -53 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-12 based upon the description of materials in all of the borings drilled within the channel segment. Intact limestone rock cores were recovered from borings EC-13-B-45, EC-13-B-47, EC-13-B-49, EC-13-B-44, EC-13-B-46 and EC-13-B-48. The Edisto Formation extends to depths greater than -62.0 feet MLLW based upon the existing drilling logs. Borings that were advanced by SPT or vibracore characterize the unit as consisting of coquina, silty calcareous sand, and cemented sand with some limestone gravel. These materials are directly correlated to the limestone recovered in the adjacent rock cores. SPT N-values indicate that the limestone is generally soft and weakly cemented, and that the material within the dredging prism are generally medium dense to very dense. The available subsurface data indicates that limestone bedrock will be encountered throughout much of the proposed dredging prism from station 311+00 to station 280+00. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface.

5.7.13. Entrance Channel, Segment EC-13

A total of 7 borings and 6 washprobes were used to describe the subsurface conditions within segment EC-13 in cross-sectional profile, as shown in Figure B-63. Boring EC-13-B-54 was used for each profile in order to extend the length of the fence diagrams within EC-13. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -52 feet MLLW, with occasional troughs that have depths up to -54 feet MLLW. The average depth along the northern fence profile is -48 feet MLLW, while the southern fence profile is deeper at -50 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-13 based upon the materials recovered from the borings drilled within the channel segment. Intact limestone rock cores were recovered from borings EC-

13-B-50, EC-13-B-52, and EC-13-B-51. Borings EC-13-B-53 and EC-13-B-54 encountered quartz sands that appeared to overlie sand mixed with weakly cemented limestone gravel. The lack of cementation in the quartz sand may indicate either a facies change within the Edisto Formation, or a poorly defined lithologic boundary between the limestone of the Edisto Formation, and the sands of the Marks Head Formation. Washprobe refusal depths seems to indicate that there is a distinctly denser surface at -52.7 to -52.8 feet MLLW, which corresponds with depth to which the limestone gravel occurs in borings EC-13-B-53 and EC-13-B-54. Therefore, the top of rock surface for the Edisto Formation is considered to lie at -52.7 feet MLLW, which is stratigraphically overlain by the medium dense sands of the Marks Head Formation. This stratigraphic positioning of units is consistent with the work of Weems and Lemon (1993), and projects the top of the Edisto Formation to gently plunge into the subsurface with increasing distance seaward. SPT N-values taken within the Edisto Formation indicate that the limestone is weakly cemented and has medium density against penetration. The sands of the Marks Head Formation, present from station 225+00 seaward are also medium dense. The available subsurface data indicates that limestone bedrock will be encountered within the proposed dredging prism from station 260+00 to at least station 210+00; however, the top of limestone bedrock surface will likely plunge from the existing bathymetric surface to -54.5 feet MLLW, and continue into the subsurface further offshore.

5.7.14. Entrance Channel, Segment EC-14

A total of 2 borings and 7 washprobes used to describe the subsurface conditions within segment EC-14 in cross-sectional profile, as shown in Figure B-64. The lack of borings within EC-14 limits the length and control by which fence diagrams can be drafted. Washprobes between the two borings were used to provide vertical control on the interpreted top of rock surface. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -54 feet MLLW. The average depth along both northern and southern fence profiles is -51.5 feet MLLW. Variations in the bathymetric depth along profile are not shown. Borings EC-13-B-54 and EC-13-B-55 encountered weakly cemented sand and limestone gravel at -54.9 and -55.6 respectively. Nearby washprobes WP-129, WP-202, WP-131, WP-203 and WP-127 have similar refusal depths that range between -54 to -56 MLLW. This suggests there is a dense cemented horizon that corresponds to the gravelly strata in borings EC-13-B-54 and EC-13-B-55. Therefore, the top of rock surface for the Edisto Formation is considered to lie between -54 and -56 feet MLLW within EC-14. Overlying the Edisto Formation is a medium dense, poorly graded quartz sand that grades seaward into an interbedded sequence of sand and silt, as shown in the borings. This material is tentatively considered part of the Marks Head Formation, based largely on the work of Weems and Lemon (1993). Little is known of this material between the two available borings EC-13-B-54 and EC-13-B-55. SPT N-values indicate that material within the dredging prism is weakly cemented and medium dense to dense. The available subsurface data indicates that weakly cemented limestone bedrock could encountered within the proposed dredging prism at -54 feet MLLW, however its horizontal extent is not well constrained because it is only controlled by two borings.

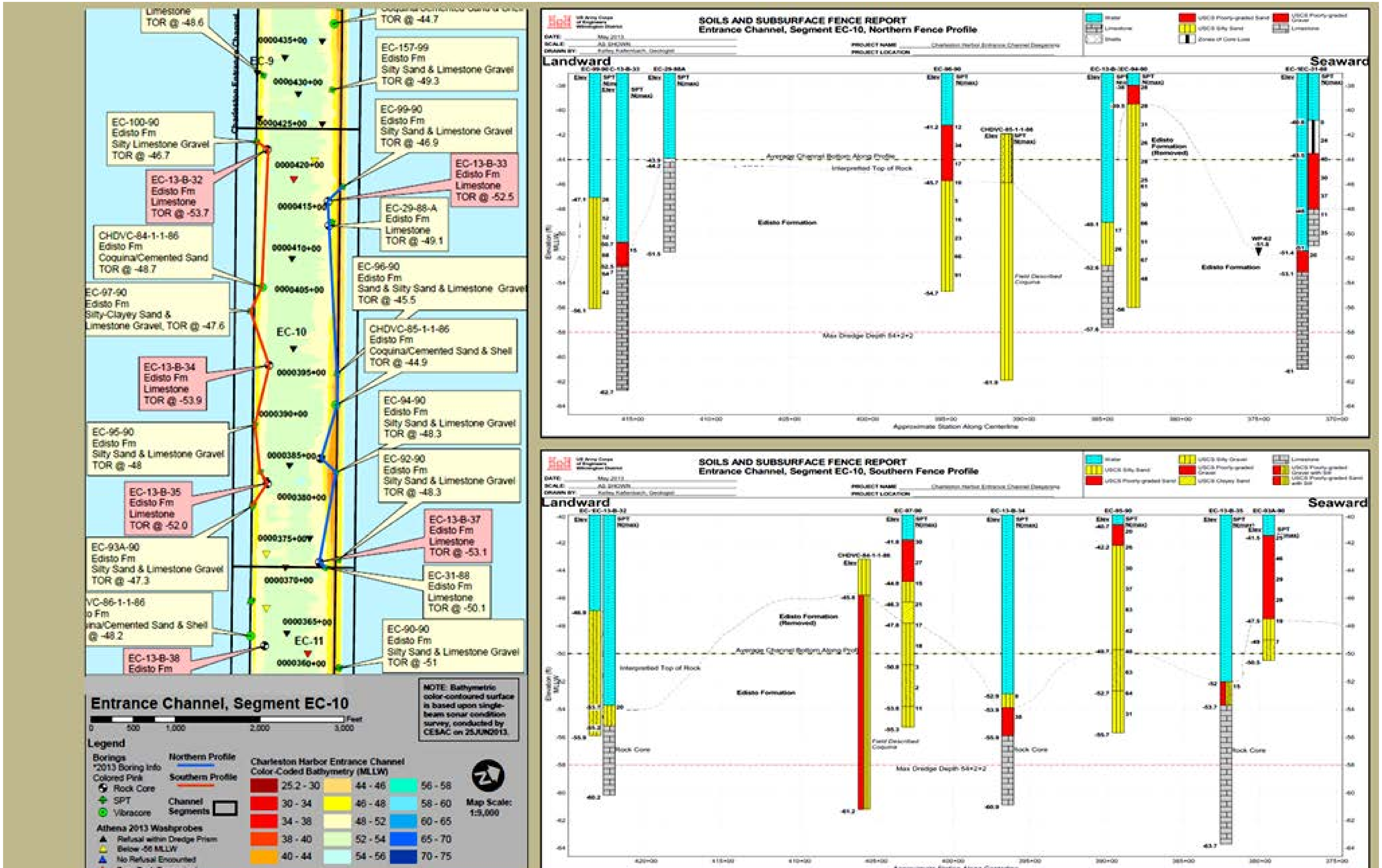


Figure B-51. Fence Diagram of Entrance Channel, Segment EC-10

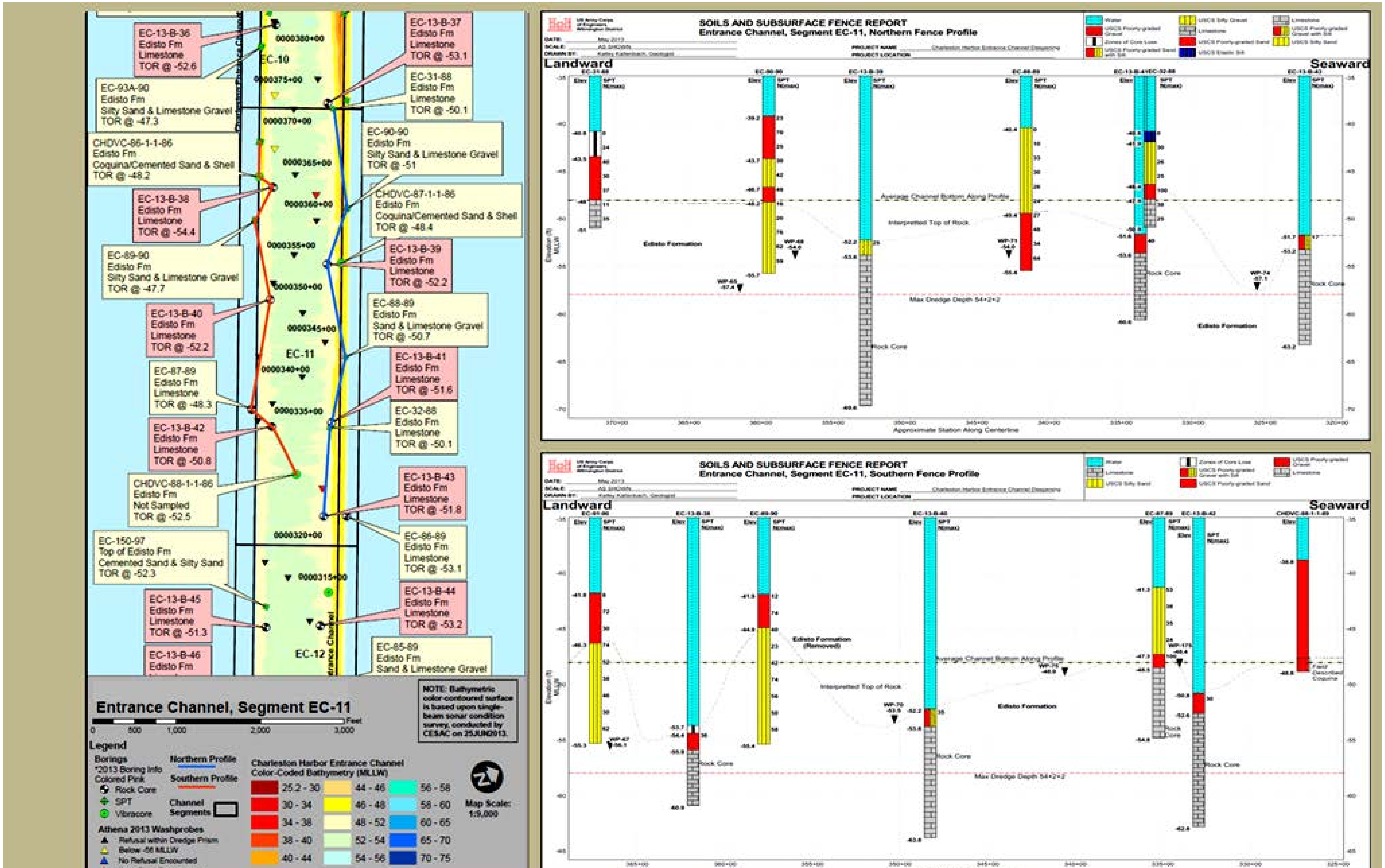


Figure B-52. Fence Diagram of Entrance Channel, Segment EC-11

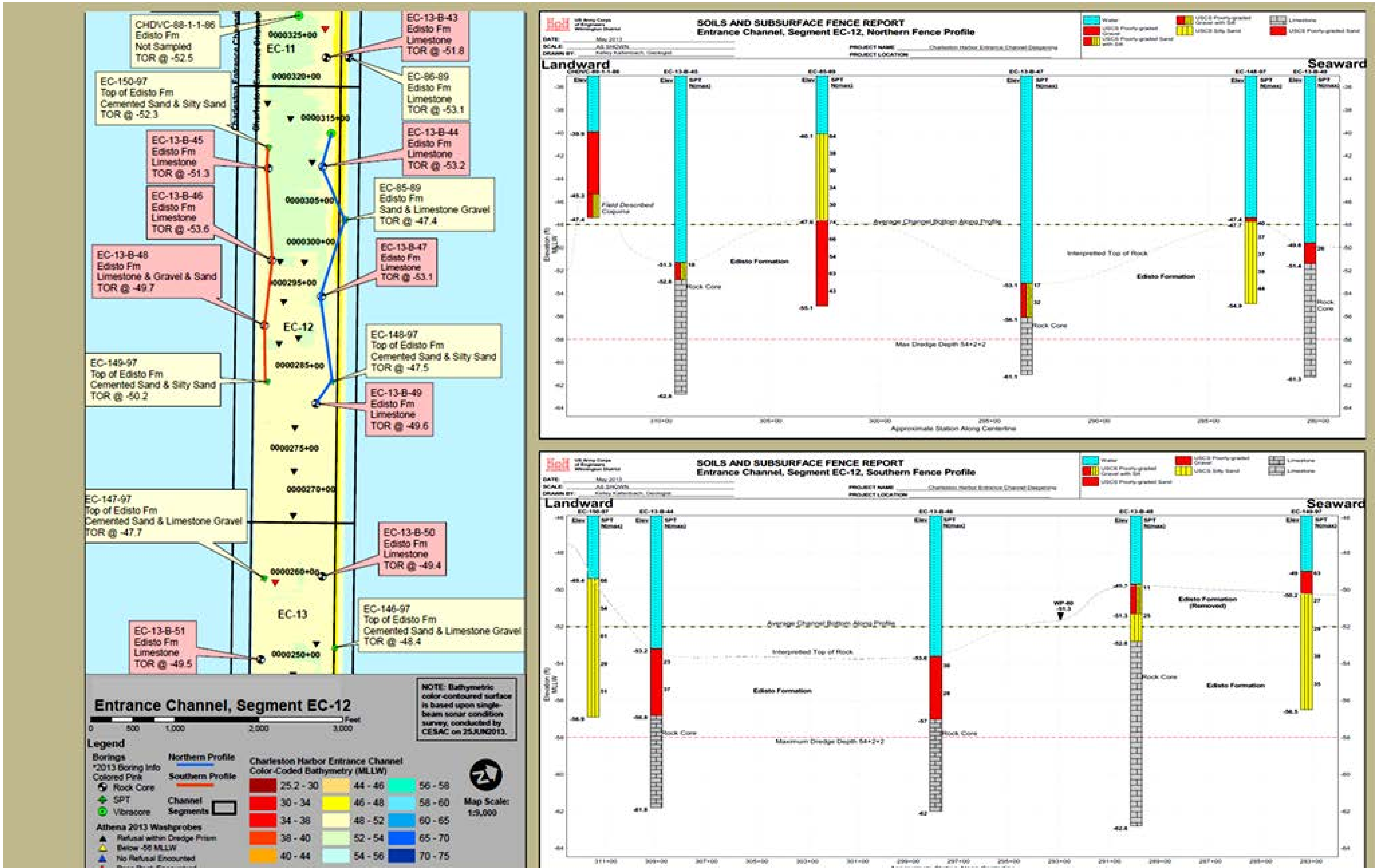


Figure B-53. Fence Diagram of Entrance Channel, Segment EC-12

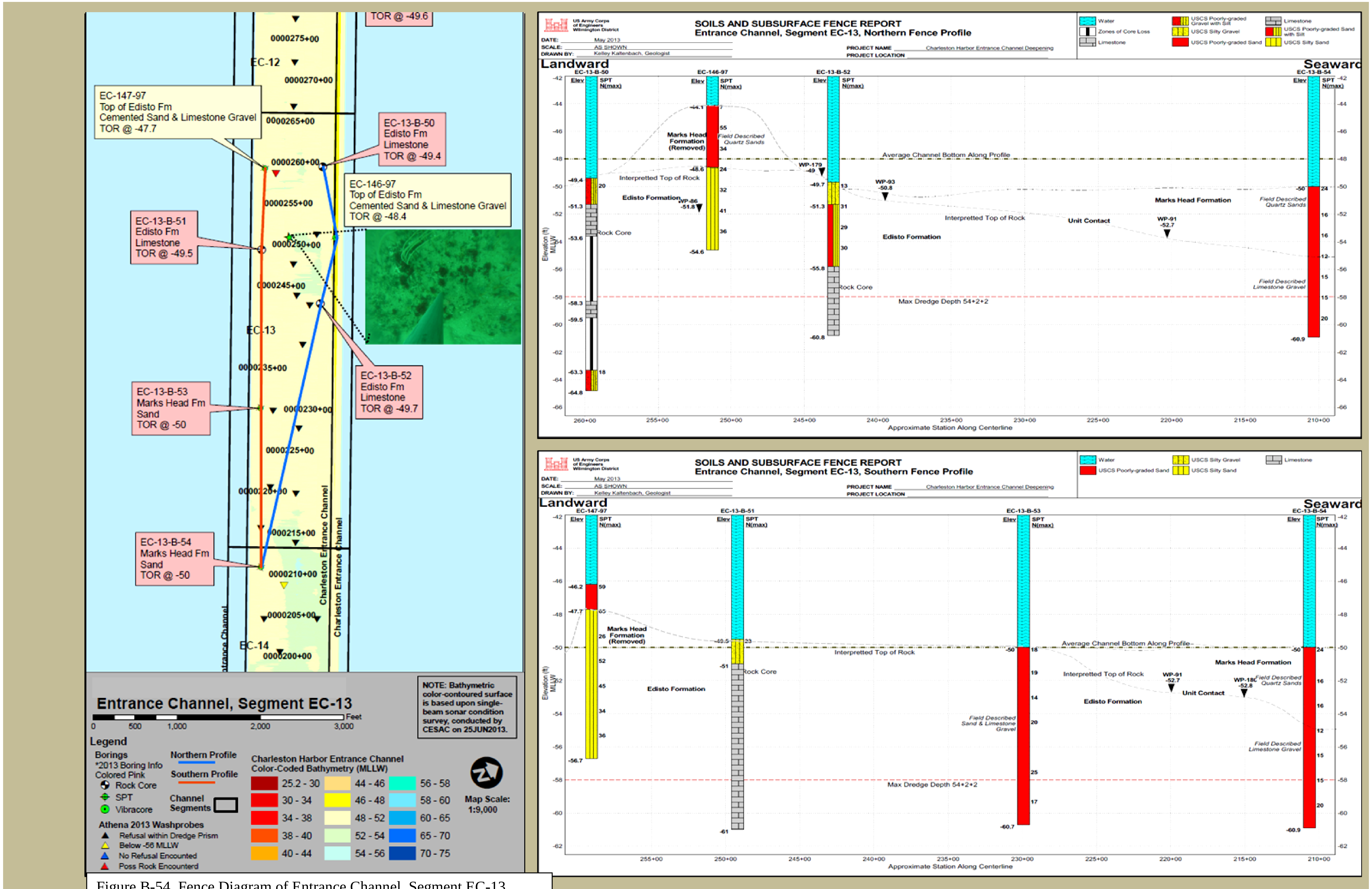


Figure B-54. Fence Diagram of Entrance Channel, Segment EC-13

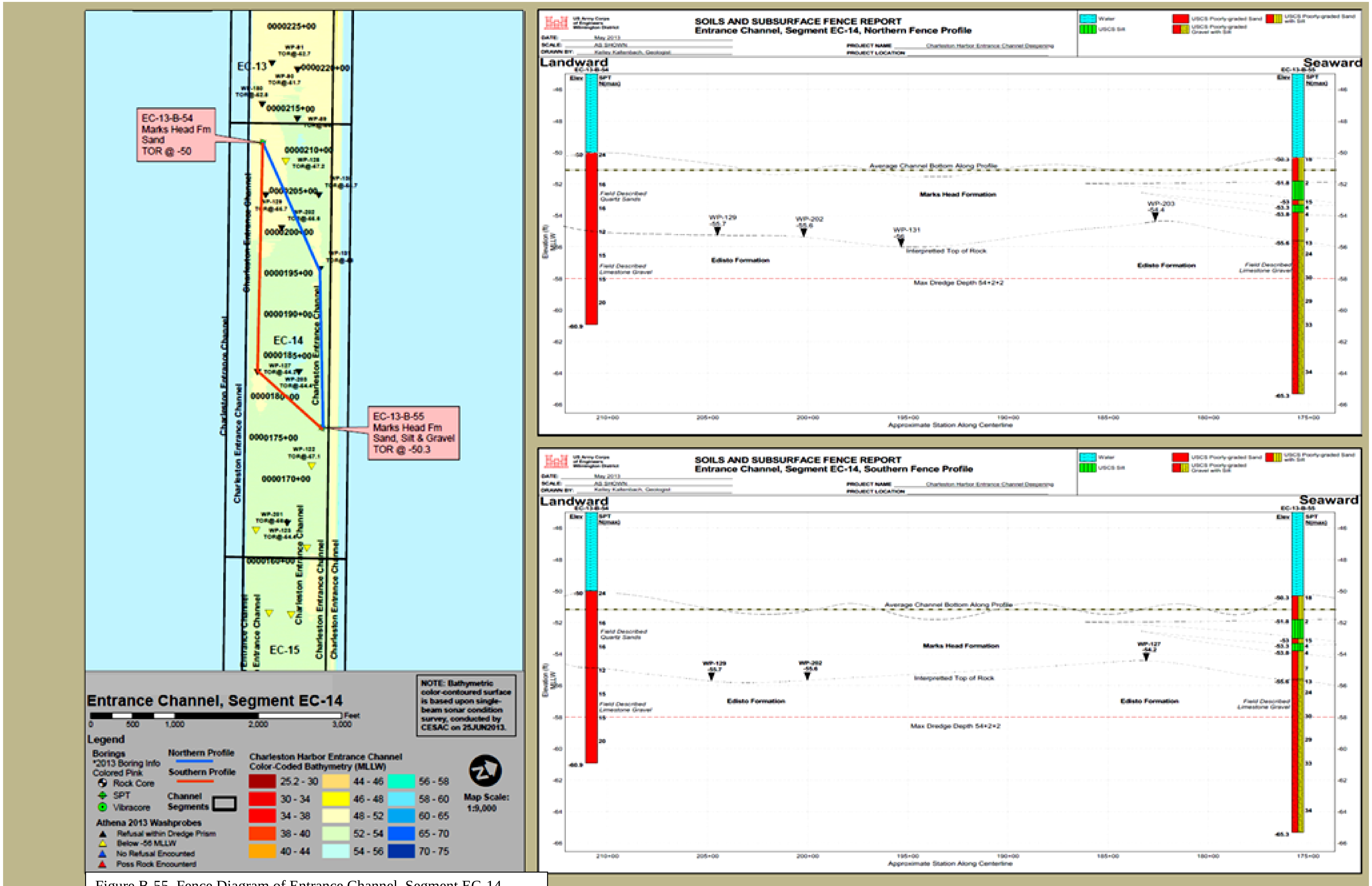


Figure B-55. Fence Diagram of Entrance Channel, Segment EC-14

5.7.15. Entrance Channel, Segment EC-15

A total of 2 borings and 13 washprobes used to describe the subsurface conditions within segment EC-15 in cross-sectional profile, as shown in Figure B-65. The lack of borings within EC-15 required the use of borings EC-13-B-55 and EC-145-97, which are located within adjacent channel segments, in order to effectively draft the fence diagrams for Figure B-65. Vertical control on the interpreted top of rock surface was augmented by the relatively abundant number of washprobes in EC-15. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -52 to -54 feet MLLW. The average depth along both northern and southern fence profiles is -52.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Boring and washprobe data suggests that the top of the Edisto Formation dips below the proposed dredging prism near station 160+00 and plunges deeper into the subsurface with increasing distance seaward. The overlying interbedded sequence of silt and sand strata, presumably part of the Marks Head Formation, appears to grade laterally into a thick bed of fat clay, bases upon material sampled in boring EC-145-97. It is not known if this material represents a facies change within the Marks Head Formation or an in-filled paleo-fluvial channel. There are no SPT N-values between the two borings in Figure B-65, however washprobe refusal is well below the maximum proposed dredge depth seaward of station 160+00, which indicates that the in-situ material is weak and can be easily removed.

5.7.16. Entrance Channel, Segment EC-16

A total of 5 borings and 9 washprobes used to describe the subsurface conditions within segment EC-16 in cross-sectional profile, as shown in Figure B-66. Boring EC-145-97 was used as a common starting point for drafting the two fence diagrams. Vertical control on the interpreted top of rock surface was augmented by the adjacent washprobes. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -58 feet MLLW. The average depth along both northern and southern fence profiles is -51.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Boring and washprobe data suggests that the top of the Edisto Formation is irregular and hummocky, but is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum consists of soft fat clay overlain by dense to very dense quartz sand, based upon the SPT borings. The dense to very dense sand occurs near station 85+00 and extends to station 60+00 on the north side of the channel. On the south side of the channel, the sand occurs near station 92+00 and extends to station 64+00. Much of the very dense sand appears to have been removed through previous harbor deepening, however the depth and lateral extent of the material is not well constrained due to the relatively few borings present in the outer channel segments. It is assumed, based upon washprobe refusal data and existing bathymetry that the dense cemented sands are limited in extent and locally comprise the banks on either side of the channel, which lie between the -48 to -52 contours (Figure B-66). This material is not as expansive as the limestone of the Edisto Formation, but may require some limited removal by rock cutter head.

5.7.17. Entrance Channel, Segment EC-17

A total 7 washprobes used to illustrate the interpreted top of rock surface within segment EC-17 in cross-sectional profile, as shown in Figure B-67. Single beam sonar condition survey dated 25JUN13 indicates that the channel bottom is extremely varied, having a bathymetric range between -48 to -70 feet MLLW. The average depth along both northern and southern fence profiles is -51.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Washprobe refusal data indicates that the interpreted top of rock surface lies near -65 feet MLLW, which is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum was penetrated by the washprobes shown in Figure B-67, therefore it is assumed that this material is very soft/loose and may be easily removed.

5.7.18. Entrance Channel, Segment EC-18

A total 6 washprobes used to illustrate the interpreted top of rock surface within segment EC-18 in cross-sectional profile, as shown in Figure B-68. Single beam sonar condition survey dated 25JUN13 indicates that the channel bottom is extremely varied, having a bathymetric range between -48 to -65 feet MLLW. The average depth along both northern and southern fence profiles is -53.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Washprobe refusal data indicates that the interpreted top of rock surface lies between -65 and -61 feet MLLW, which is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum was penetrated by the washprobes shown in Figure B-68, therefore it is assumed that this material is very soft/loose and may be easily removed.

5.7.19. Entrance Channel, Segment EC-19

A total 8 washprobes used to illustrate the interpreted top of rock surface within segment EC-19 in cross-sectional profile, as shown in Figure B-69. Single beam sonar condition survey dated 25JUN13 indicates that the channel bottom is extremely varied, having a bathymetric range between -48 to -65 feet MLLW. The average depth along both northern and southern fence profiles is -53.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Washprobe refusal data indicates that the interpreted top of rock surface lies between -64 and -61 feet MLLW, which is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum was penetrated by the washprobes shown in Figure B-69, therefore it is assumed that this material is very soft/loose and may be easily removed.

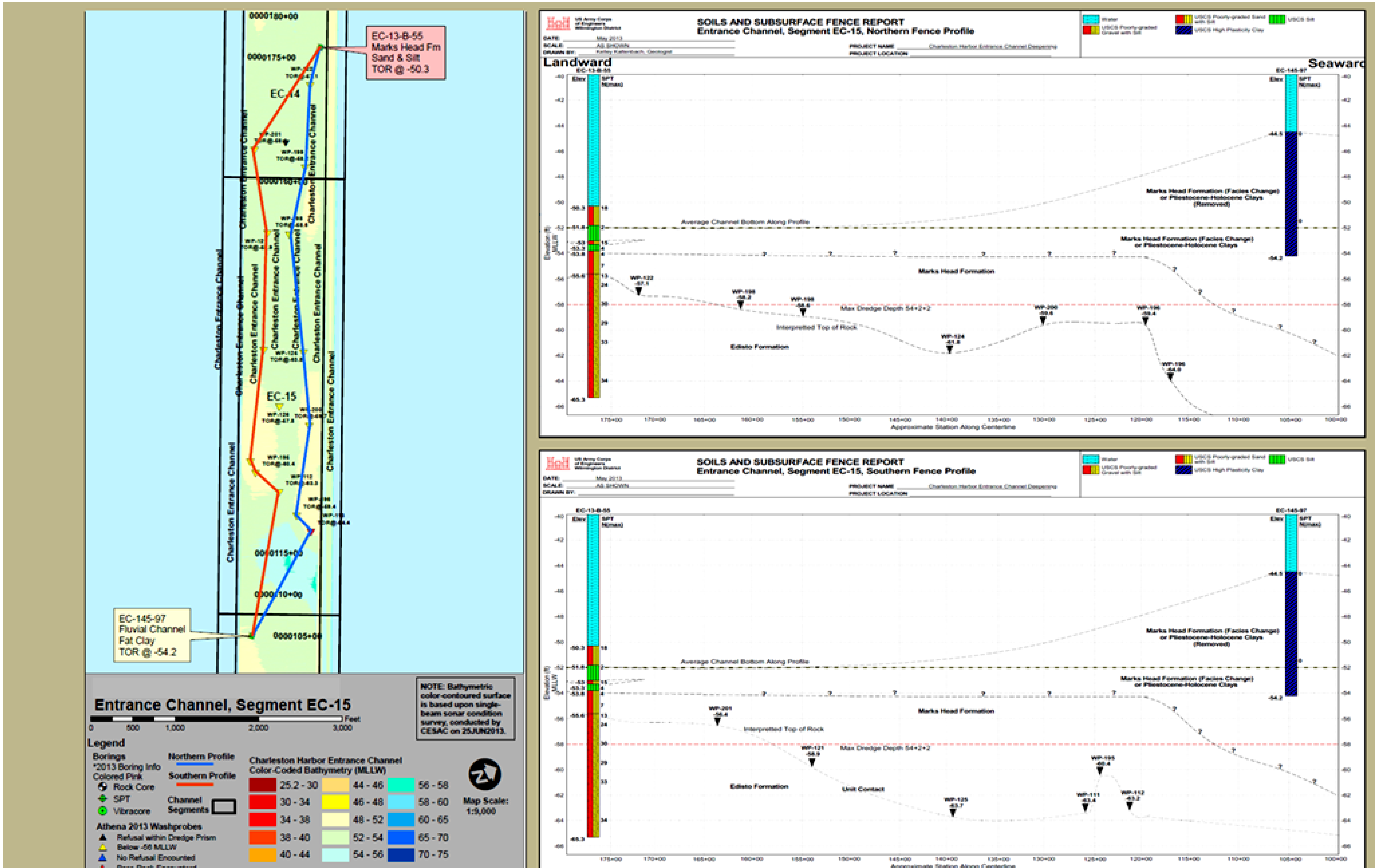


Figure B-56. Fence Diagram of Entrance Channel, Segment EC-15

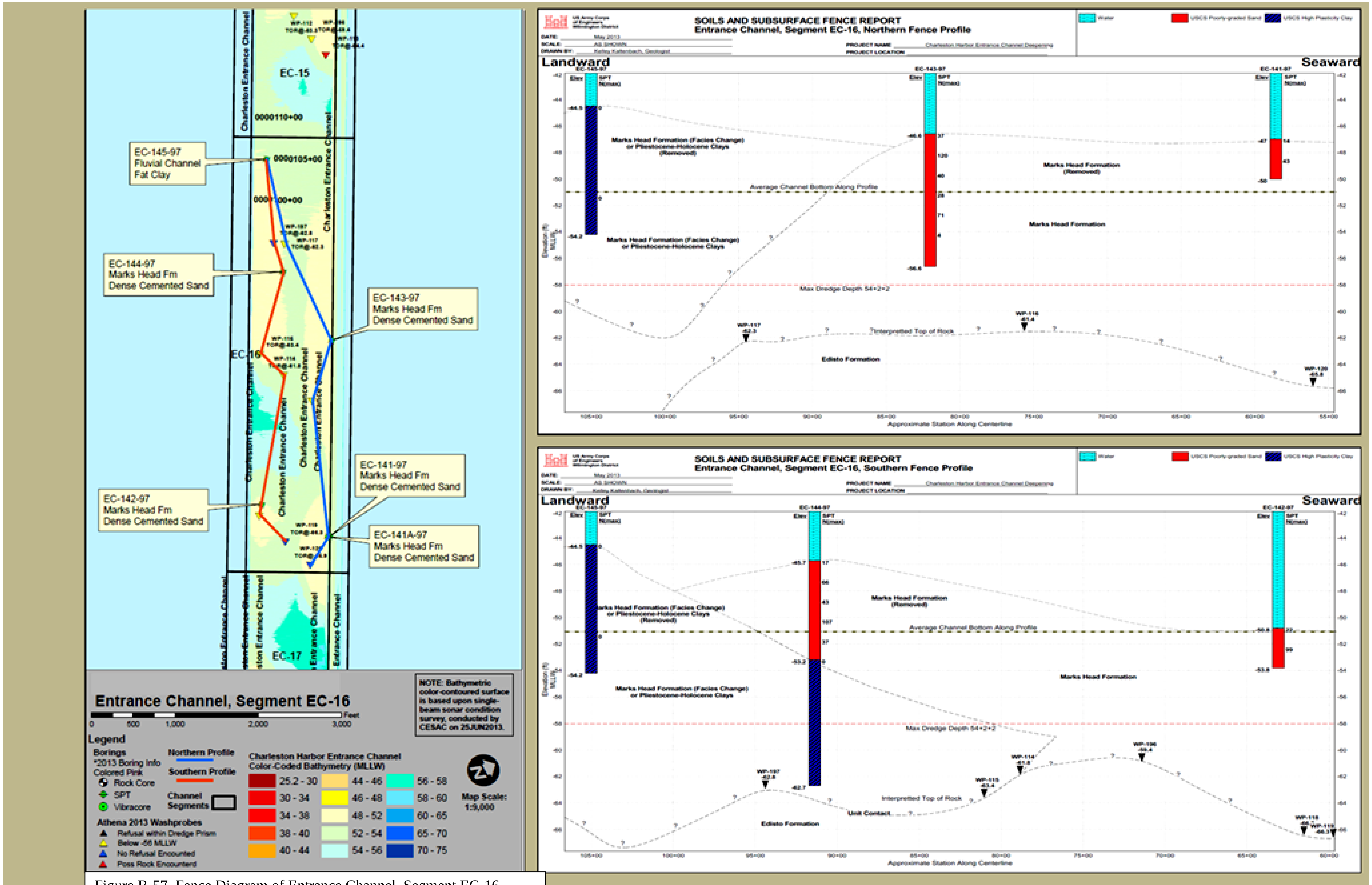


Figure B-57. Fence Diagram of Entrance Channel, Segment EC-16

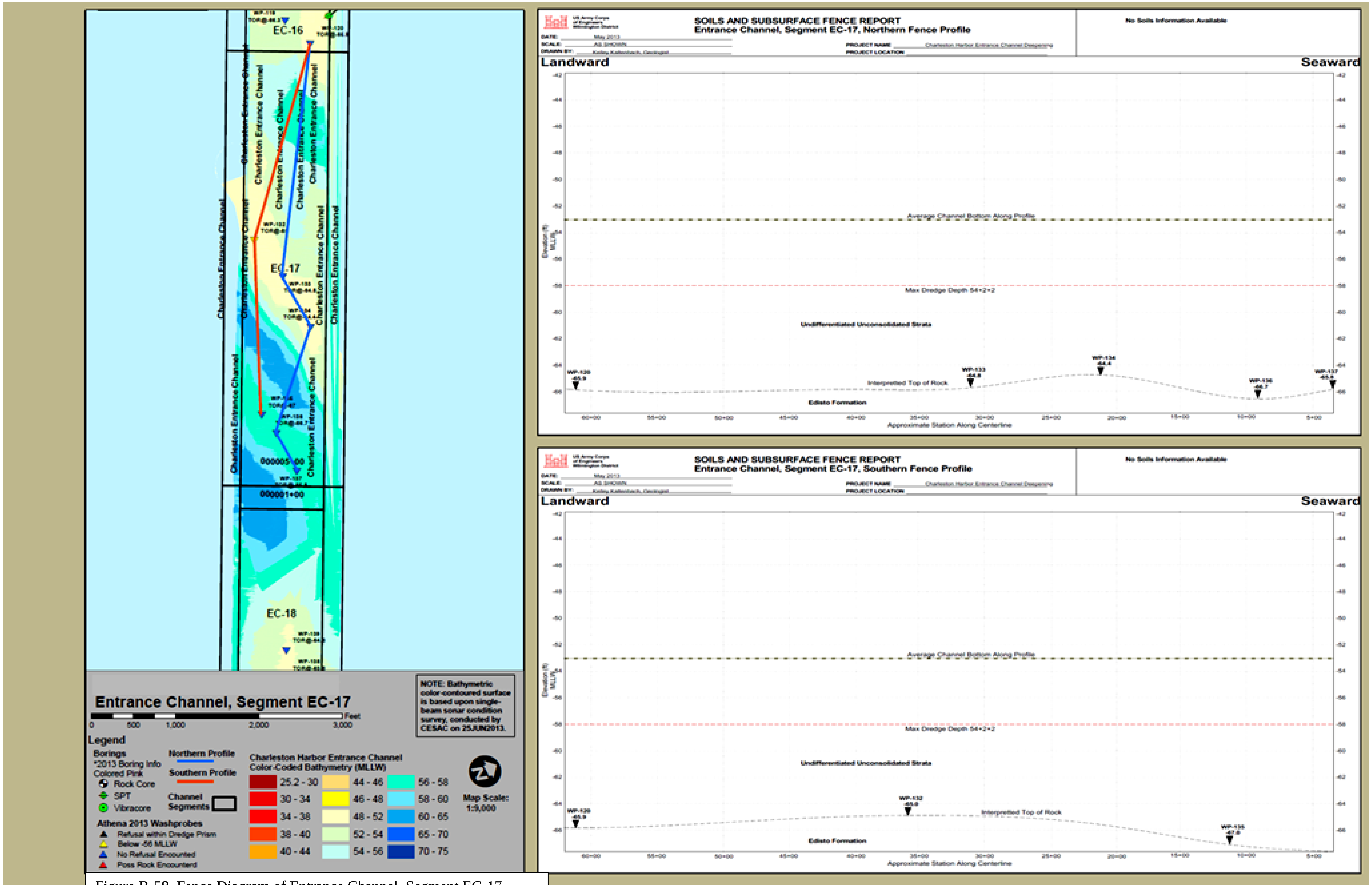


Figure B-58. Fence Diagram of Entrance Channel, Segment EC-17

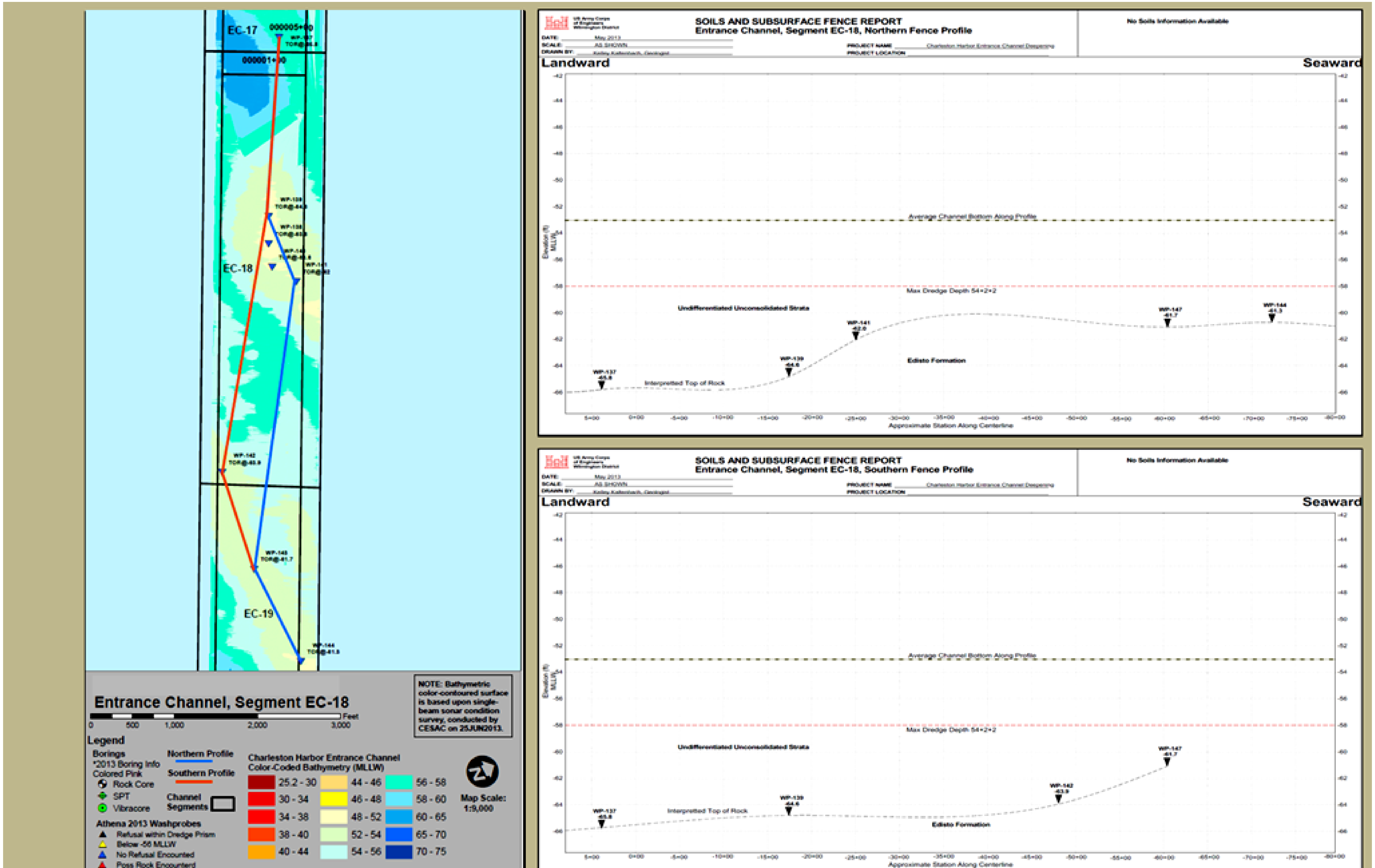


Figure B-68. Fence Diagram of Entrance Channel, Segment EC-18

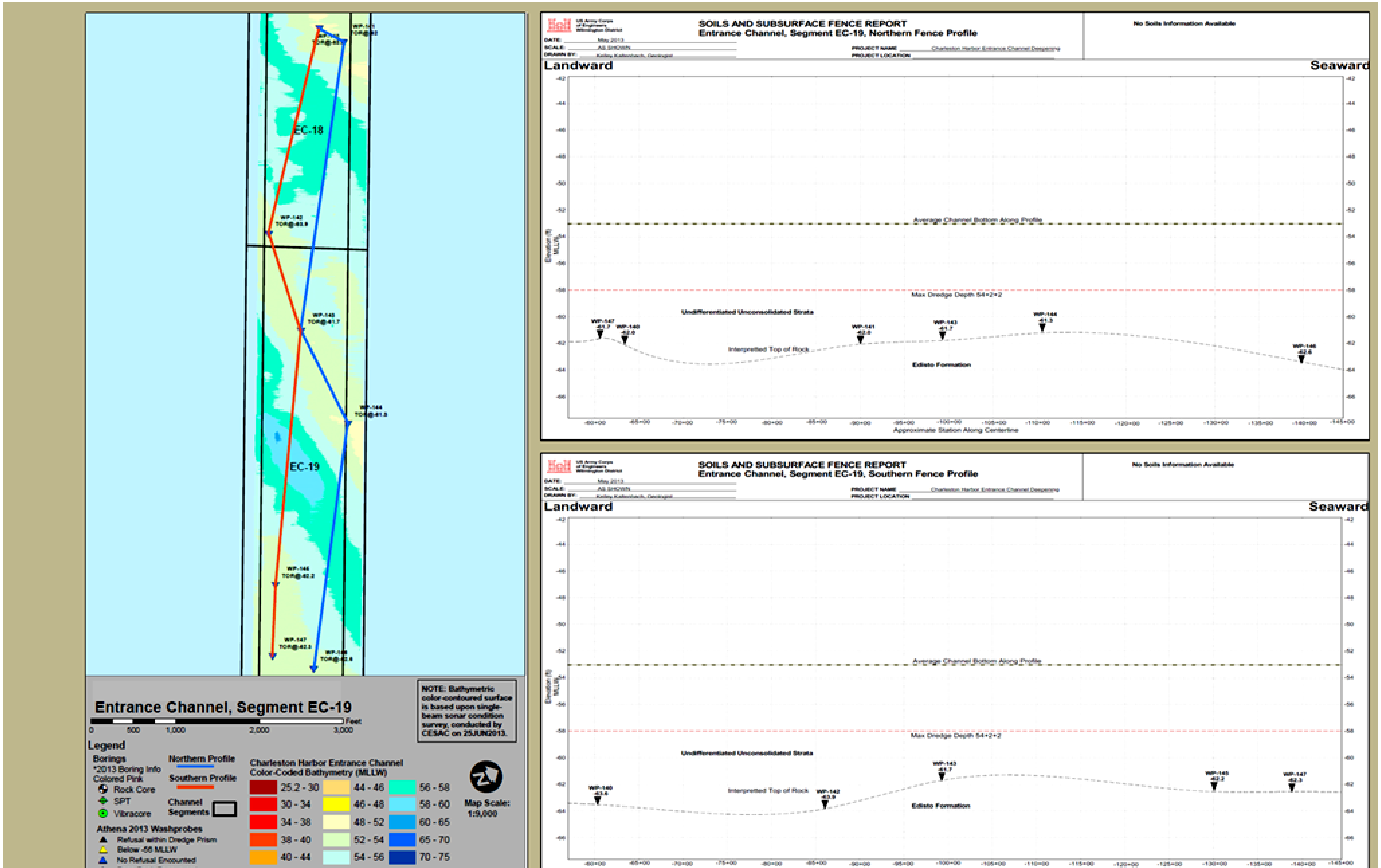


Figure B-69. Fence Diagram of Entrance Channel, Segment EC-19

5.7.20 Stratigraphic Summary

A summary table that shows the predominant geologic materials that can be expected to be encountered if the channel is deepened to -58 feet MLLW is shown below. SPT N-values for fine-grained and granular material are listed for reference.

Table B-15. Entrance Channel Stratigraphic Summary

Figure	Reach	Predominant Material	SPT-N (fine-grained)	SPT-N (granular)
B-51	Entrance Channel, EC-1	Inorganic Silt, Clayey Sand	2 - 16	0 - 19
B-52	Entrance Channel, EC-2	Inorganic Silt, Clayey Sand	0 - 18	1 - 81
B-53	Entrance Channel, EC-3	Inorganic Silt, Fat Clay, Silty Sand	5 - 12	3 - 12
B-54	Entrance Channel, EC-4	Inorganic Silt, Silty Sand	7 - 12	5 - 14
B-55	Entrance Channel, EC-5	Silty Sand, Sand, Limestone, Silt	4 - 9	8 - 46
B-56	Entrance Channel, EC-6	Limestone, Clayey-Silty Sand, Sand	---	15 -40
B-57	Entrance Channel, EC-7	Limestone, Silty Sand, Sand, Silt	2 - 4	6 - 42
B-58	Entrance Channel, EC-8	Limestone, Silty-Clayey Sand, Sand	---	3 - 29
B-59	Entrance Channel, EC-9	Limestone, Fat Clay, Silty Sand	0 - 5	11 - 100
B-60	Entrance Channel, EC-10	Limestone, Silty Sand, Sand	---	2 - 91
B-61	Entrance Channel, EC-11	Limestone, Silty Sand, Sand	---	11 – 76
B-62	Entrance Channel, EC-12	Limestone, Silty Sand, Sand	---	18 – 74
B-63	Entrance Channel, EC-13	Limestone, Sand	---	12 - 36
B-64	Entrance Channel, EC-14	Sand, Gravel	---	12 - 30
B-65	Entrance Channel, EC-15	Sand, Gravel, Silt, Clay	0 - 4	7 – 30
B-66	Entrance Channel, EC-16	Fat Clay, Sand	0	22 - 99
B-67 to 69	Entrance Channel EC-17 to 19	No material data available	Assume < 2	Assume < 4

5.8 Mapping and Volume Estimates of Limestone within the Entrance Channel

5.8.1. Geologic Strip Map

The subsurface materials encountered during drilling vary laterally along the length of the entrance channel, as well as vertically. The lateral distribution of sediments roughly corresponds to the stratigraphic framework and geologic mapping of the Charleston area by Weems and Lemon (1993). A geologic strip map was initially developed using the 2013 boring data, because it was during the drilling operations in which the full extent of the Edisto Formation in the channel was recognized. The intact limestone rock cores can be correlated to previous investigations where the geologist characterized disarticulated limestone recovered from SPT drilling as a gravel or sand. The limestone is largely based upon a silty sand matrix with variable amounts of shell, which is consistent with previous workers descriptions. Given this correlation, the historical data was then re-analyzed and used refine the unit boundaries. A revised geologic strip map (Plate 12) was then developed that combines both 2013 and historical drilling data shows the lateral variation of geologic materials within the entrance channel.

Limestone bedrock belonging to the Edisto Formation occurs within channel segments EC-4 through EC-13 (see Plate 12). Drilling records (Attachment B-2) indicate that there are lesser amounts of limestone along the northern sides of channel segments EC-6 and EC-7. What may be interpreted as northerly trending paleofluvial channel system is incised into the limestone bedrock within EC-5, EC-6, and EC-7 (see Plate 4, Plate 12, Figures B-58 to B-59). The majority of the limestone is located within channel segments EC-5, EC-7 and EC-8 through EC-12.

5.8.2. Area Dimensions

The estimated area and maximum thickness of limestone bedrock within the proposed dredging prism is provided in the table below. The thickness estimates include cemented granular soils such as limestone gravels; this material is interpreted to be top of limestone bedrock.

Table B-16. Maximum dimensions of rock per segment based drilling data.

Channel Segment	Area (sq. feet)	Max Thickness (feet)
EC-4	1,114,646	2.5
EC-5	4,145, 692	12.9
EC-6	2,188, 318	7.3
EC-7	3,028,295	6.6
EC-8	4,500, 286	10.0
EC-9	5,433,416	11.2
EC-10	5,560,563	6.6
EC-11	5,759,802	7.2
EC-12	5,756,055	8.4
EC-13	3,720,418	8.6

5.8.3. Revised Rock Volume Estimate

The results from the 2013 drilling program were used to revise the excavation rock volumes to facilitate better project cost estimation. The method used to calculate the new work rock volume requires that the geometries of the top of rock (TOR) and the proposed channel prism be subtracted from each other by 3-D vector analysis using Hypack, Microstation, or ArcGIS software.

Wilmington District, USACE created a composite TOR dataset that combined the historical drilling data with the washprobe and rock cores drilled in 2013. The dataset was formatted as an XYZ point data set where the easting and northing coordinates of the source borings represent the X and Y values accordingly, and the elevation of TOR represents the Z value. Each drilling record had to meet screening criteria before it was used order to build TOR point dataset. Entrance channel borings were visually scanned for descriptions that contained limestone, coquina, limestone gravel, calcareous sand, cemented sand, and shelly sand, which is recognized as an indicator of material belonging to the Edisto Formation. Once recognized, these borings were separated and a set of principles were applied to establish top of rock elevations for each data point;

- TOR = elevation of top of rock within borings
- TOR = elevation at which limestone gravel is first recognized in the boring
- TOR = Bathymetric surface in historical borings that contain calcareous soils and gravels that extend above the present (25JUN13) bathymetric condition survey.
- TOR = completion elevation in borings that lie within boundaries of the Edisto Formation, but may have been drilled within paleo-fluvial channels that are incised into the limestone.

These principles are conservative, because the natural TOR surface may be deeper or less well defined, but they were necessary in order to maintain the data density required to build the TOR surface. The TOR dataset (Attachment B-5) was then given to USACE-Charleston District for computational analysis. SAC personnel conducted several iterations of volume calculations using ArcGIS and Hypack software separately in order to assure quality control. The results of the volume calculations are presented in Table B-17. The majority of the rock lies within segments EC-4 to EC-13. The total volume of rock that is estimated to need removal for a -58 foot MLLW channel is 9,698,919 cubic yards. This estimate is 2-3 times greater than the original estimate of 3,476,646 cubic yards, but is considered more accurate because the geology of the channel is much better defined.

Table B-17. Revised volume estimates of limestone within the entrance channel.

Estimated Material Quantities Undifferentiated (CY)		PRE-2013 EXPLORATION-HISTORICAL RECORDS ONLY	% Type Material Within -58 MLLW Dredgiing Prism (Based Upon 1986-1999 Borings)				Initial 2012-2013 Estimate Rock Volume CY	POST- 2013 EXPLORATION DRILLING & TOP OF ROCK SURFACE DEVELOPMENT	2014 Revised Rock Volume Estimate		
	58'		% Unconsolidated	% Soft Rock	% Hard Rock	% Unknown			Total Rock Calculated (58')	Rock Above Condition	Rock Needing Removal (58')
Segment 1	569,596		76%	0%	0%	24%	0		0	0	0
Segment 2	435,529		58%	17%	5%	19%	98,720		0	0	0
Segment 3	625,978		59%	7%	0%	34%	44,713		0	0	0
Segment 4	737,540		35%	52%	0%	14%	380,117		1,482,956	238,272	**1,244,684**
Segment 5	729,419		46%	34%	11%	9%	329,509		1,167,207	9,809	**1,157,398**
Segment 6	652,831		52%	38%	0%	10%	249,584		863,488	10,370	**853,118**
Segment 7	573,134		62%	33%	0%	5%	187,686		972,260	65,274	**906,986**
Segment 8	507,662		54%	35%	6%	5%	208,271		878,613	57,003	**821,610**
Segment 9	476,307		38%	24%	34%	3%	279,830		1,074,904	202,113	**872,791**
Segment 10	550,547		30%	16%	47%	7%	347,359		1,175,070	167,258	**1,007,812**
Segment 11	517,333		17%	5%	73%	5%	405,458		1,013,277	63,134	**950,143**
Segment 12	450,290		18%	30%	52%	0%	368,809		1,355,248	186,918	**1,168,330**
Segment 13	430,406		17%	33%	50%	0%	358,671		741,992	25,945	**716,047**
Segment 14	287,713		0%	0%	0%	100%	0		0	0	0
Segment 15	289,292		0%	0%	0%	100%	0		0	0	0
Segment 16	367,736		35%	31%	28%	6%	217,918		0	0	0
Segment 17	188,858		0%	0%	0%	100%	0		0	0	0
Segment 18	118,868		0%	0%	0%	100%	0		0	0	0
Segment 19	147,116		0%	0%	0%	100%	0		0	0	0
Segment 20	108,614		0%	0%	0%	100%	0		0	0	0
Segment 21	2,470		0%	0%	0%	100%	0		0	0	0
Total QTY (CY)	**8,767,238**						**3,476,646**		10,725,015	1,026,096	**9,698,919**

5.9 Summary of Lab Testing

5.9.1. Soil Test Results

Attachment B-3 contains the material gradation data and lab results. A summary of these results is provided in Table B-18. The majority of the materials submitted for testing were granular in nature, while only 15 samples were fine-grained. The laboratory visual classification of granular materials tended to be finer grained than the field visual classification. This difference is likely due to a number of factors; field biases in the observation of the material, subsequent desiccation of granular soils, mechanical breaking of intergranular cemented bonds during test preparation and sieving, etc.

Table B-18. Summary of 2013 Entrance Channel Material Properties from USACE-EMU.

Lab Number	Hole Number	Sample Number	Depth (ft) MLLW	D6913 % Passing No.4 %	No 200 %	D4318 Atterberg Limits LL	PL	PI	D2216 MC%	Color	Class Symbol	D2487 Unified Soil Classification System
K2/3289	EC-13-B-1	1	52.0 to 53.5	96.8	52.8	44	31	13	23.0	Very Dark Grayish Brown	ML	Sandy Inorganic Silt Low LL (ML), with a trace of gravel.
K2/3292	EC-13-B-1	4	56.5 to 58.0	100.0	22.9	41	36	5	40.6	Dark Olive Gray	SM	Silty Sand (SM).
K2/3297	EC-13-B-2	3	55.9 to 57.4	100.0	53.3	50	45	5	33.3	Very Dark Grayish Brown	MH	Sandy Inorganic Silt High LL (MH).
K2/3301	EC-13-B-3	2	57.3 to 58.8	100.0	52.4	47	41	6	30.0	Dark Olive Gray	ML	Sandy Inorganic Silt Low LL (ML).
K2/3303	EC-13-B-3	4	60.3 to 61.8	99.4	27.6	---	---	---	36.4	Dark Olive Gray	SM	Silty Sand (SM).
K2/3306	EC-13-B-4	2	55.5 to 57.0	100.0	15.4	---	---	---	35.9	Very Dark Gray	SM	Silty Sand (SM).
K2/3308	EC-13-B-4	4	59.2 to 60.7	100.0	24.9	---	---	---	37.1	Very Dark Gray	SM	Silty Sand (SM).
K2/3310	EC-13-B-4	6	62.2 to 63.7	100.0	51.8	---	---	---	35.6	Dark Olive Gray	ML	(Visual) Sandy Inorganic Silt Low LL (ML).
K2/3316	EC-13-B-5	2	52.9 to 54.4	100.0	33.4	---	---	---	48.3	Black	SM	Silty Sand (SM).
K2/3318	EC-13-B-5	4	55.9 to 57.4	100.0	19.0	---	---	---	39.0	Black	SM	Silty Sand (SM).
K2/3320	EC-13-B-5	6	58.9 to 60.4	99.7	19.8	---	---	---	36.9	Black	SM	Silty Sand (SM).
K2/3322	EC-13-B-6	2	52.3 to 53.8	100.0	29.6	63	43	20	38.1	Black	SM-H	Silty Sand High LL (SM-H).
K2/3323	EC-13-B-6	3	54.3 to 55.8	100.0	33.2	75	58	17	48.1	Black	SM-H	Silty Sand High LL (SM-H).
K2/3325	EC-13-B-6	5	57.3 to 58.8	100.0	23.7	---	---	---	42.0	Black	SM	Silty Sand (SM).
K2/3330	EC-13-B-7	2	53.4 to 54.9	100.0	30.4	---	---	---	37.7	Black	SM	Silty Sand (SM).
K2/3332	EC-13-B-7	4	56.7 to 58.2	100.0	21.5	---	---	---	36.4	Black	SM	Silty Sand (SM).
K2/3335	EC-13-B-7	7	61.8 to 63.3	100.0	23.0	---	---	---	36.6	Very Dark Gray	SM	Silty Sand (SM).
K2/3338	EC-13-B-8	2	54.2 to 55.7	100.0	55.4	---	---	---	41.3	Black	MH	(Visual) Sandy Inorganic Silt High LL (MH).
K2/3340	EC-13-B-8	4	57.2 to 58.7	100.0	30.6	64	49	15	33.0	Black	SM-H	Silty Sand High LL (SM-H).
K2/3342	EC-13-B-8	6	60.2 to 61.7	100.0	21.0	---	---	---	41.4	Black	SM	Silty Sand (SM).
K2/3345	EC-13-B-9	2	52.9 to 54.4	100.0	79.4	96	52	44	51.7	Very Dark Gray	MH	Inorganic Silt High LL (MH), with some sand.
K2/3347	EC-13-B-9	4	55.9 to 57.4	100.0	73.9	---	---	---	53.8	Black	MH	(Visual) Inorganic Silt High LL (MH), with some sand.
K2/3349	EC-13-B-9	6	58.9 to 60.4	100.0	58.5	---	---	---	54.1	Black	MH	(Visual) Sandy Inorganic Silt High LL (MH).
K2/3351	EC-13-B-9	8	61.9 to 63.4	100.0	39.4	94	63	31	46.8	Black	SM-H	Silty Sand High LL (SM-H).
K2/3355	EC-13-B-10	3B	52.3 to 52.6	58.7	19.4	---	---	---	25.3	Olive	SM	Gravelly Silty Sand (SM).
K2/3356	EC-13-B-10	4	53.1 to 54.6	100.0	87.7	132	40	92	52.1	Very Dark Gray	CH	Fat Clay (CH), with a little sand.
K2/3358	EC-13-B-10	6	56.6 to 58.1	95.5	72.0	119	68	51	59.7	Dark Olive Gray	MH	Inorganic Silt High LL (MH), with some sand and a trace of gravel.
K2/3361	EC-13-B-10	9	61.7 to 63.2	100.0	40.0	---	---	---	45.1	Black	SM	Silty Sand (SM).
K2/3364	EC-13-B-11	2	55.8 to 57.3	86.7	28.3	---	---	---	30.2	Olive	SM	Silty Sand (SM), with a little gravel.
K2/3365	EC-13-B-11	3	57.3 to 58.8	78.3	25.6	---	---	---	35.5	Olive	SM	Silty Sand (SM), with some gravel.
K2/3366	EC-13-B-11	4	58.8 to 60.3	100.0	59.6	74	33	41	49.7	Very Dark Gray	CH	Sandy Fat Clay (CH).
K2/3369	EC-13-B-12	2	53.8 to 55.3	97.4	21.4	---	---	---	34.1	Olive Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3371	EC-13-B-12	4	56.8 to 58.3	90.4	23.2	---	---	---	30.4	Olive Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3373	EC-13-B-12	6	59.8 to 61.3	99.5	40.6	---	---	---	40.8	Olive Gray	SC	(Visual) Clayey Sand (SC).
K2/3374	EC-13-B-12	7	61.3 to 62.8	100.0	89.9	100	32	68	54.4	Very Dark Gray	CH	Fat Clay (CH), with a little sand.
K2/3376	EC-13-B-13	2	51.6 to 53.1	98.5	19.6	---	---	---	24.5	Gray & Light Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3380	EC-13-B-13	6	57.7 to 59.2	98.3	15.8	---	---	---	31.2	Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3381	EC-13-B-13	7	59.2 to 60.7	91.2	14.7	---	---	---	31.6	Gray	SM	Silty Sand (SM), with a trace of gravel.

5.9.2. Rock Testing Results

Attachment B-4 contains the laboratory rock strength data sheets. A summary of this testing is provided in Table B-19. A total of 65 unconfined compressive strength tests were run once on each of the submitted core samples. The minimum and maximum UC strengths encountered were 73.7 psi and 415.8 psi respectively. The average UC strength is 162.5 psi. A total of 80 Brazilian splitting tensile strength tests were run on the samples submitted, in addition to duplicates cut from untested UC-sample trimmings. The minimum and maximum tensile strength encountered were 0.7 psi and 136 psi. The average rock tensile strength is 37.1 psi, which is 23% or roughly a quarter of the average UC strength.

Table B-19. Summary of 2013 Entrance Channel Rock Strength Testing from USACE-EMU

Lab Number	Boring #	Sample #	Elevation Interval	Test	Diameter	UCS (psi)	STS-A (psi)	STS-B (psi)	STS-C (psi)
K2/3203	EC-13-B-28	1	53.4-53.7	STS	HQ		11.0		
3204	EC-13-B-28	2	54.1-54.6	UCS	HQ	88.8			
3205	EC-13-B-28	3	57.0-57.5	UCS	HQ	97.6			
3206	EC-13-B-28	4	57.7-58.1	UCS	HQ	95.2			
3207	EC-13-B-28	5	58.8-59.3	UCS	HQ	56.7			
3208	EC-13-B-28	6	59.5-59.8	STS	HQ		19.1	19.9	18.5
3209	EC-13-B-32	1	55.3-55.6	STS	HQ		64.7	76.0	61.5
3210	EC-13-B-32	2	56.0-56.5	UCS	HQ	189.4			
3211	EC-13-B-32	3	58.1-58.6	UCS	HQ	249.7			
3212	EC-13-B-33	1	53.1-53.5	UCS	HQ	350.9			
3213	EC-13-B-33	2	55.0-55.4	UCS	HQ	237.8			
3214	EC-13-B-33	3	56.0-56.4	STS	HQ		37.9		
3215	EC-13-B-33	4	58.5-58.9	UCS	HQ	322.1			
3216	EC-13-B-34	1	56.4-56.8	STS	HQ		14.8		
3217	EC-13-B-34	2	57.7-58.2	UCS	HQ	124.7			
3218	EC-13-B-34	3	59.7-60.2	UCS	HQ	194.6			
3219	EC-13-B-35	1	53.7-54.1	STS	HQ		2.5	10.5	
3220	EC-13-B-35	2	55.0-55.5	UCS	HQ	195.0			
3221	EC-13-B-35	3	59.0-59.5	UCS	HQ	231.0			
3222	EC-13-B-36	1	54.3-54.8	UCS	HQ	183.9			
3223	EC-13-B-36	2	56.7-57.2	UCS	HQ	145.4			
3224	EC-13-B-37	1	53.6-53.9	STS	HQ		15.7		
3225	EC-13-B-37	2	55.3-55.8	STS	HQ		24.0	11.2	
3226	EC-13-B-37	3	59.2-59.7	UCS	HQ	174.5			
3227	EC-13-B-38	1	56.2-56.7	UCS	HQ	33.3			
3228	EC-13-B-38	2	57.7-58.0	STS	HQ		34.1	26.5	11.8
3229	EC-13-B-38	3	59.0-59.5	UCS	HQ	100.7			
3230	EC-13-B-39	1	54.2-54.7	UCS	PQ	176.5			
3231	EC-13-B-39	2	55.2-55.7	STS	PQ		59.0	89.8	50.4
3232	EC-13-B-39	3	57.2-57.7	UCS	PQ	248.9			
3233	EC-13-B-39	4	58.7-59.3	UCS	PQ	253.3			
3234	EC-13-B-39	5	59.3-59.8	STS	PQ		31.3	64.5	37.7
3235	EC-13-B-40	1	53.7-54.3	UCS	PQ	295.5			
3236	EC-13-B-40	2	55.8-56.3	UCS	PQ	292.9			
3237	EC-13-B-40	3	56.7-57.7	STS	PQ		70.8	56.7	66.5
3238	EC-13-B-40	4	58.7-59.3	UCS	PQ	232.1			
3239	EC-13-B-41	1	53.6-54.1	UCS	PQ	186.0			
3240	EC-13-B-41	2	55.9-56.4	UCS	PQ	226.3			
3241	EC-13-B-41	3	57.4-57.8	STS	PQ		36.6	77.1	86.2
3242	EC-13-B-41	4	58.6-59.0	STS	HQ		40.9	74.3	33.9
3243	EC-13-B-41	5	59.5-60.0	UCS	HQ	273.7			
3244	EC-13-B-42	1	53.0-53.5	UCS	PQ	223.3			
3245	EC-13-B-42	2	54.6-55.1	UCS	PQ	195.2			
3246	EC-13-B-42	3	55.8-56.1	STS	PQ		31.8	22.6	
3247	EC-13-B-42	4	57.9-58.4	UCS	PQ	200.1			
3248	EC-13-B-42	5	59.3-59.6	STS	PQ		60.4	70.1	82.5

Lab Number	Boring #	Sample #	Elevation Interval	Test	Diameter	UCS (psi)	STS-A (psi)	STS-B (psi)	STS-C (psi)
3249	EC-13-B-43	1	54.0-54.5	UCS	PQ	369.2			
3250	EC-13-B-43	2	55.4-55.8	STS	PQ		63.2	56.3	36.6
3251	EC-13-B-43	3	56.6-57.1	UCS	PQ	415.8			
3252	EC-13-B-43	4	58.3-58.8	UCS	PQ	219.3			
3253	EC-13-B-43	5	59.3-59.7	STS	PQ		136.0	113.5	112.4
3254	EC-13-B-44	1	56.8-57.3	UCS	PQ	114.6			
3255	EC-13-B-44	2	58.4-58.8	STS	PQ		40.7	17.7	21.3
3256	EC-13-B-44	3	59.4-59.9	UCS	PQ	158.7			
3257	EC-13-B-45	1	53.7-54.2	UCS	PQ	227.4			
3258	EC-13-B-45	2	55.0-55.5	STS	PQ		31.7	26.8	32.1
3259	EC-13-B-45	3	55.8-56.3	UCS	PQ	200.5			
3260	EC-13-B-45	4	57.8-58.3	UCS	PQ	191.4			
3261	EC-13-B-45	5	59.5-60.0	STS	PQ		24.4	52.2	
3262	EC-13-B-46	1	57.5-58.0	UCS	PQ	138.4			
3263	EC-13-B-46	2	59.0-59.5	STS	PQ		2.8	42.8	56.2
3264	EC-13-B-46	3	59.9-60.4	UCS	PQ	170.5			
3265	EC-13-B-47	1	56.1-56.7	UCS	PQ	130.5			
3266	EC-13-B-47	2	57.2-57.7	STS	PQ		22.2		
3267	EC-13-B-47	3	58.5-59.0	UCS	PQ	152.3			
3268	EC-13-B-48	1	52.7-53.2	UCS	PQ	98.4			
3269	EC-13-B-48	2	52.9-53.4	UCS	PQ	204.9			
3270	EC-13-B-48	3	57.1-57.6	STS	PQ		13.6		
3271	EC-13-B-48	4	57.7-58.2	UCS	PQ	89.1			
3272	EC-13-B-48	5	59.7-60.2	UCS	PQ	142.4			
3273	EC-13-B-48	6	58.7-59.2	STS	PQ		38.9	30.3	55.6
3274	EC-13-B-49	1	53.1-53.7	UCS	PQ	84.8			
3275	EC-13-B-49	2	55.7-56.2	UCS	PQ	88.1			
3276	EC-13-B-49	3	56.6-56.9	STS	PQ		8.4		
3277	EC-13-B-49	4	58.4-58.9	UCS	PQ	0.0			
3278	EC-13-B-50	1	51.6-52.1	UCS	HQ	115.3			
3279	EC-13-B-50	2	53.2-53.6	UCS	HQ	73.7			
3280	EC-13-B-50	3	58.3-58.6	STS	HQ		22.8	26.5	18.1
3281	EC-13-B-51	1	51.5-51.9	UCS	PQ	76.4			
3282	EC-13-B-51	2	52.9-53.4	UCS	PQ	77.0			
3283	EC-13-B-51	3	54.2-54.7	STS	PQ		19.0		
3284	EC-13-B-51	4	56.0-56.6	UCS	HQ	95.3			
3285	EC-13-B-51	5	58.4-58.7	STS	HQ		20.8		
3286	EC-13-B-52	1	57.9-58.4	UCS	PQ	107.2			
3287	EC-13-B-52	2	59.8-60.3	UCS	PQ	101.0			
3288	EC-13-B-52	3	57.0-57.4	STS	PQ		13.4	18.7	17.6
3502	EC-13-B-18	1	53.9-54.4	UCS	HQ	139.8			
3503	EC-13-B-18	2	55.0-55.3	STS	HQ		11.5	6.9	10.4
3504	EC-13-B-18	3	57.3-57.8	UCS	HQ	139.1			
3505	EC-13-B-18	4	58.6-58.9	STS	HQ		26.6		
3506	EC-13-B-18	5	59.4-59.9	UCS	HQ	122.4			
3507	EC-13-B-20	1	57.2-53.2	UCS	HQ	209.9			
3508	EC-13-B-20	2	55.6-56.0	STS	HQ		5.1		
3509	EC-13-B-20	3	57.2-57.6	STS	HQ		10.6	4.3	
3510	EC-13-B-20	4	58.7-59.2	UCS	HQ	154.7			
3511	EC-13-B-21	1	53.5-54.0	UCS	HQ	120.3			
3512	EC-13-B-21	2	54.9-55.2	STS	HQ		18.2		
3513	EC-13-B-21	3	56.0-56.5	UCS	HQ	150.8			
3514	EC-13-B-21	4	57.9-58.4	UCS	HQ	158.0			
3515	EC-13-B-21	5	59.1-59.4	STS	HQ		29.1	12.1	0.7
3516	EC-13-B-24	1	56.0-56.5	UCS	PQ	77.4			
3517	EC-13-B-24	2	57.5-58.0	UCS	PQ	79.7			
3518	EC-13-B-24	3	58.5-58.8	STS	PQ		21.8		
3519	EC-13-B-24	4	59.5-59.8	STS	PQ		14.5		
3287	EC-13-B-52	2	59.8-60.3	UCS	PQ	101.0			
3288	EC-13-B-52	3	57.0-57.4	STS	PQ		13.4	18.7	17.6
3502	EC-13-B-18	1	53.9-54.4	UCS	HQ	139.8			
3503	EC-13-B-18	2	55.0-55.3	STS	HQ		11.5	6.9	10.4
3504	EC-13-B-18	3	57.3-57.8	UCS	HQ	139.1			
3505	EC-13-B-18	4	58.6-58.9	STS	HQ		26.6		

Lab Number	Boring #	Sample #	Elevation Interval	Test	Diameter	UCS (psi)	STS-A (psi)	STS-B (psi)	STS-C (psi)
3506	EC-13-B-18	5	59.4-59.9	UCS	HQ	122.4			
3507	EC-13-B-20	1	57.2-53.2	UCS	HQ	209.9			
3508	EC-13-B-20	2	55.6-56.0	STS	HQ		5.1		
3509	EC-13-B-20	3	57.2-57.6	STS	HQ		10.6	4.3	
3510	EC-13-B-20	4	58.7-59.2	UCS	HQ	154.7			
3511	EC-13-B-21	1	53.5-54.0	UCS	HQ	120.3			
3512	EC-13-B-21	2	54.9-55.2	STS	HQ		18.2		
3513	EC-13-B-21	3	56.0-56.5	UCS	HQ	150.8			
3514	EC-13-B-21	4	57.9-58.4	UCS	HQ	158.0			
3515	EC-13-B-21	5	59.1-59.4	STS	HQ		29.1	12.1	0.7
3516	EC-13-B-24	1	56.0-56.5	UCS	PQ	77.4			
3517	EC-13-B-24	2	57.5-58.0	UCS	PQ	79.7			
3518	EC-13-B-24	3	58.5-58.8	STS	PQ		21.8		
3519	EC-13-B-24	4	59.5-59.8	STS	PQ		14.5		

5.10 Rock Dredgeability

5.10.1. Parameters used to Determine Rock Dredgeability by Rock Cutter-Head

USACE-Wilmington District used the following rock strength parameters to determine rock dredgeability; unconfined compressive strength, splitting tensile strength, percent core recovery, rock quality designation, and the thickness of bedrock. Of these parameters, it has been the collective experience[19] within Wilmington District that the unconfined compressive strength of the rock plays the greatest role in the determination of its dredgeability.

The unconfined compressive strength of rock is one of the most widely regarded indicators of rock dredgeability (USACE, 1983; Hignett, 1984; Smith, 1987, 1994; Bieniawski, 1989; Vervoort and DeWitt, 1997). These workers have indicated through their individual fields of expertise that the UCS is the best indicator of material dredgeability. Hignett (1984) reported that the maximum unconfined compressive strength that rock cutter head dredges could effectively remove ranged from 3625 psi to 4351 psi, even though their individual components were rated for much stronger rock. These figures were given for 1970’s to 1980’s era dredges, which have probably been upgraded in capacity in the 30 years since the publication. The other parameters become increasingly important when strong rock is encountered and the dredging contractor must alter his plan of work in order to utilize natural planes of weakness within the rock for economic removal. Above 4351 psi, the rock must be blasted to allow removal (Hignett, 1984).

In the case of the Wilmington Harbor Anchorage Basin, the average unconfined compressive strength of the in-situ rock was 548 psi, with a strength range from 301 psi to 1364 psi. The Anchorage Basin was assessed by the Wilmington District to be dredgeable, but there were initial concerns to rock dredgeability in areas that had rock strengths in excess of 500 psi and thicknesses greater than 4-feet (Figure B-70). Great Lakes Dock and Dredging mobilized the *D/B Texas* to the site in December 2012 and removed all of the rock in the Anchorage Basin without the need for blasting. The rock mass in the area of concern was removed easily without incident.

[19] Based on rock dredging experience from Wilmington Harbor, which has much harder limestone than Charleston Harbor. Specific rock dredging projects include the Baldhead Shoals Re-alighnment and Anchorage Basin Deepening. Coastal southeastern NC has similar geology as Charleston, SC, but the bedrock is much better cemented. Wilmington Harbor could be considered a more extreme case in terms of rock strength and cementation, than Charleston Harbor.

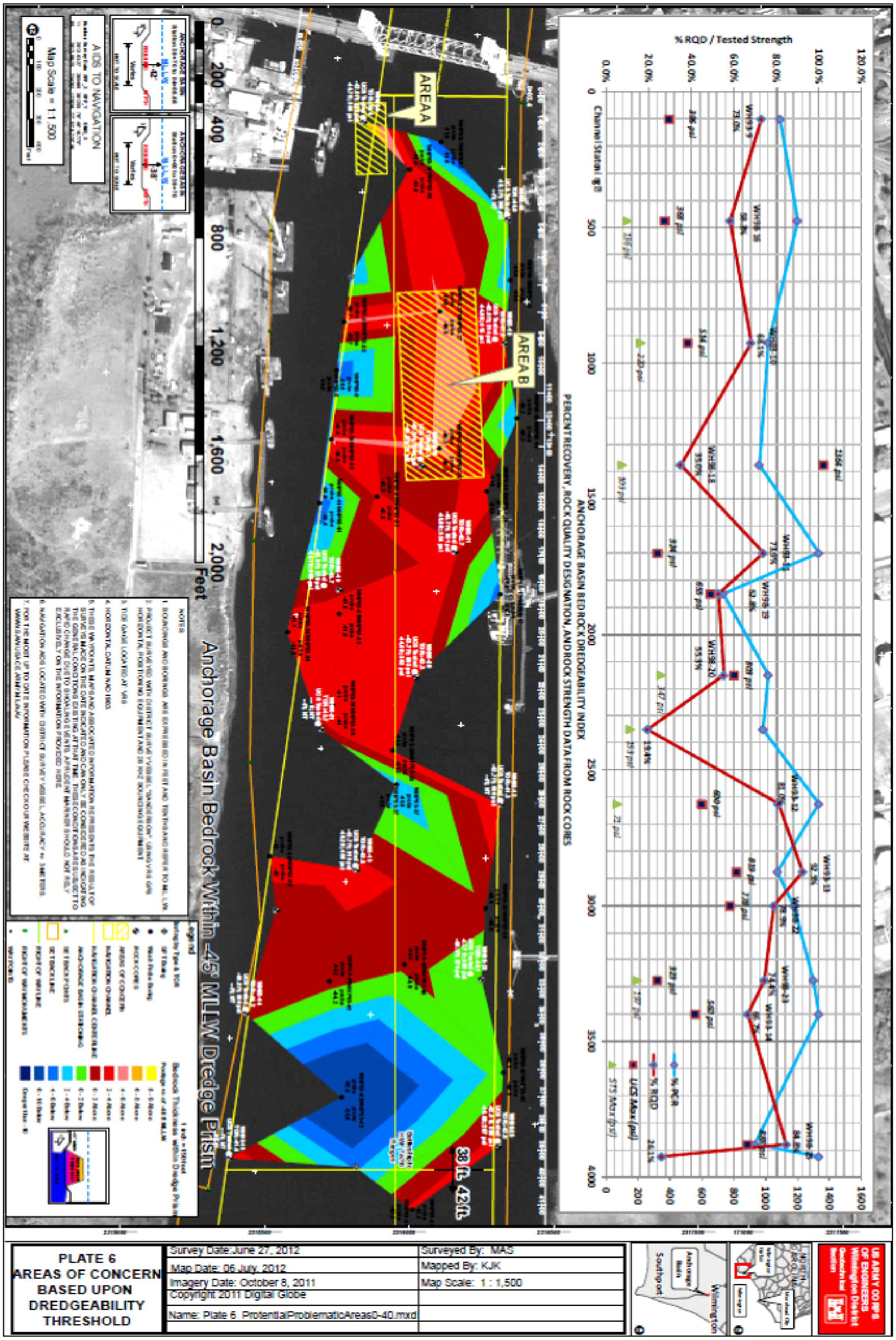


Figure B-59. Wilmington Harbor Anchorage Basin problematic areas > 500 psi & > 4-feet thick.

5.10.2. Strength of Materials within the Entrance Channel

The strength of the material sampled during the 2013 drilling program was tabulated in Excel, and plotted against the existing maps, as illustrated in Plates 13 and 14. The maximum N-blow count from all SPT sampling (1988 to 2013) is plotted against channel stationing for segments EC-1 through EC-16. SPT N-values for the recent drilling are plotted in red, while the historical SPT values are plotted in dark blue. The maximum unconfined compressive strength (UCS) of limestone samples taken within the dredging prism (< -58 MLLW) are plotted as red point data, alongside historical UCS test data from USACE (black) and GLDD (gray).

The Cooper Formation floors much of channel segments EC-1 into most of EC-4 (Figure B-54 through Figure B-56). This fine-grained, silty-clayey material is medium stiff to very stiff based upon SPT N-values that range from 4 to 19. No limestone was encountered within channel segments EC-1 through EC-3. The materials in these segments, though consolidated to some degree, are not cemented and should be considered low-strength. Historical data indicates that the limestone may occur as thin, discontinuous beds within EC-4.

Transitional sand or paleofluvial material floor the northern side of channel segments EC-5, EC-6, EC-7 and a small portion of EC-8 (Figure B-57 through Figure B-60). These materials have variable amounts of cementation and compaction, which appear to have a wide variation of relative density. The graph of SPT N-values in Plate 14 indicates that the density of these materials range from loose (N = 4) to dense (N = 40). The higher densities are considered indicative the limestone that is shown to lie along the southern bank of these channel segments. Borings along the northern bank that have relatively high blow count values may have intercepted zones of deeply indurated limestone, or coarse-grained detrital material that was shed off the limestone subcroppings along the southern bank.

Subsurface data indicates that the density and relative strength of material rises from EC-5 to EC-6. Rock sampled from these sections is no more than 210 psi in strength. A small erosional window of Cooper Marl is denoted in EC-7 where the N-value drops below 5, then jumps up to N=20 in response to the re-encountering granular material in boring EC-13-B-22. Weak limestone (98 psi) is present larger quantity near the end of EC-8.

The strength of the limestone present in channel segments EC-10 through EC-13 (Figure B-62 to Figure B-65) is less than 450 psi, based upon the results of the 2013 lab testing. When compared to the GLDD UCS data, most of the rock strengths are much weaker. The highest rock strength values are within the GLDD dataset, notably UC strengths of 994 psi and 1670 psi. However, as discussed in section 2.4.2, these values do not represent the overall strength of the rock mass, but rather the strength of isolated well-silicified, discontinuous strata, and should be considered data outliers. Therefore the strength range of the limestone bedrock is generally constrained to 450 psi or less.

Based upon the low strength of the rock within the entrance channel, and the ease by which stronger rock was removed from Wilmington Harbor's Anchorage Basin by rock cutter head alone, there should be no need for blasting in Charleston Harbor. The rock that is present should be easily removable by a modern rock cutter head dredge.

5.10.3. Seismic Vibration

Seismic vibration generated from rock cutter-head dredging should pose no risk to existing structures within Charleston. There are two lines of reasoning for this;

1. The location where rock dredging will occur is distant from any structure. Any seismic waves generated will be sufficiently attenuated below established peak particle velocity (PPV) damage thresholds. These thresholds were established by the U.S. Bureau of Mines and Reclamation, which determined that structures exposed to peak particle velocities (PPV) of 3.0 to 10.0 in/sec. would sustain damage to drywall and plaster, or cracking of masonry and concrete. For reference, it should be noted that rock dredging conducted in Wilmington Harbor was located 1-2 miles from the downtown historical district, never exceeded the established PPV threshold.

2. Foundation soils in Charleston have already been subjected to relatively high PPV's from previous large magnitude earthquakes. Foundation structures may have already settled as a result of liquefaction of the underlying non-cohesive soils (where present). Furthermore, multiple earthquake events may have induced settlement of foundation soils, effectively buffering any settlement effects (however unlikely) from the seismic waves generated from the cutter-head.

5.11 Conclusions

- The limestone previously encountered by Great Lakes Docks and Dredging belongs to the Edisto Formation and is much more widespread than initially anticipated.

- Volume estimates using TOR modeling and the proposed channel template (-58 MLLW) indicate that the volume of rock that will need to be removed is 9,698,919 cubic yards. This estimate is 2-3 times greater than the original estimate of 3,476,646 cubic yards, but is considered more accurate because the geology of the channel is much better defined.

- Overall, the unconfined compressive strength of tested samples indicates that the limestone is very weak and soft. Low unconfined compressive strength bedrock is very conducive to removal by rock cutter head dredging.

- Based upon the available drilling logs and lab data, and using conservative engineering-geology judgment, there should be no need to conduct blasting to remove bedrock.

- The need for vibration monitoring is not anticipated for this project.

VI. CLOUTER CREEK

6.1 Introduction

Clouter Creek Disposal Area (DA) is a diked upland area that is used to contain material dredged from the Cooper River for navigational purposes. It is located east of North Charleston, on the east bank of the Cooper River. The east side of Clouter Creek DA is bordered by Clouter Creek, while the north, south, and west sides are bordered by the Cooper River. Totaling roughly 1,475 acres, Clouter Creek DA is divided into four "cells", South Cell, Middle Cell, Highway Cell, and North Cell. The approximate acreages are as follows:

Table B-20.

Clouter Creek DA Area	
South Cell	415 Acres
Middle Cell	410 Acres
Highway Cell	460 Acres
North Cell	190 Acres

The portion of the Cooper River dredged material placed into Clouter Creek Disposal Area consists of the upper harbor, from the Daniel Island Reach to the Ordnance Reach. The northern third of Clouter Creek DA is owned by the South Carolina State Ports Authority (SCSPA), and the southern two-thirds are owned by the U.S. Army Corps of Engineers. The Federal Government enjoys a perpetual easement on the state owned portion.

6.2 Fifty Year Future Life Cycle

6.2.1 Current Dredging Volume

The upper harbor reaches are dredged on a bi-annual basis (every 18-24 months). The yearly dredge material average that is placed into Clouter Creek DA is 837,216 cubic yards. Authorized third party users also place dredged material into Clouter Creek DA on a yearly basis, with an average annual volume of 448,749 cubic yards. The total average annual dredged material disposal amount that is placed in Clouter Creek Disposal Area is almost 1.3 million cubic yards.

6.2.2 New Work

New work is divided into two areas: upper harbor individual reaches and wideners. The current authorized dredging depth in the upper harbor is 45-feet, plus 2 to 4-feet of advanced maintenance and an additional 2-feet allowable overdepth, for a total depth of 49-feet. The exception to this are areas of high shoaling[20] which have additional allowance for maintenance dredging. Minimum new work depth is 47-feet, plus 2-feet advanced maintenance and 2-feet allowable overdepth for a total depth of 51'. Maximum new work depth is 52-feet, plus 2-feet

[20] High shoaling areas in Lower Wando, Lower Town Creek, Ordnance Reaches, Ordnance Turning Basin, and Wando Turning Basin are required to have 45' depth with 4' of authorized advanced maintenance dredging and an additional 2' allowable overdepth. Drum Island Reach is required to have 45', plus 6' of authorized advanced maintenance, and an additional 2' allowable overdepth.

advanced maintenance and 2-feet allowable overdepth for a total of 56-feet, with additional allowance for high shoaling areas. Wideners are to be dredged to the same depth as the channel segments. Maximum new work depth is 52', plus 2' advanced maintenance and 2' allowable overdepth for a total of 56'. The new work volume of dredged material ranges from 373,481 cubic yards to almost 6 million cubic yards. See Table B-22 for individual quantities. A critical design issue for the proposed dike raises to accommodate current and new work dredging volume is settlement and stability.

Table B-21.

New Work Volume (cy)*										
Reaches	**47'**	**48'**	**49'**	**50'**	**51'**	**52'**	**53'**	**54'**	**55'**	**56'**
Daniel Island Reach				125,375	300,709	519,440	773,667	1,041,015	1,314,719	1,592,690
Daniel Island Bend				15,962	37,045	74,551				
Clouter Creek Reach				96,155	232,407	389,959				
Navy Yard Reach				81,661	211,072	358,816				
N Charleston Reach				33,372	109,877	225,645				
Fiblin Creek Reach				23,387	69,348	156,072				
Port Terminal Reach				27,374	78,918	160,376				
Ordinance Reach						30,989	72,331	118,091		
Ordinance Reach Turning Basin						56,845	116,170	176,617		
Wideners (Maximum Option)	**47'**	**48'**	**49'**	**50'**	**51'**	**52'**	**53'**	**54'**	**55'**	**56'**
Daniel Island Reach			386,121	411,412	451,556	478,874	499,692	527,341	548,115	576,062
Clouter Creek Reach	77,292	97,588	119,837	143,280	167,650	193,191				
N Charleston Reach	163,555	189,374	216,743	245,331	276,119	307,048				
Fiblin Creek Reach	117,449	140,583	165,494	192,131	220,283	249,348				
Fiblin-Port Terminal Intersection	15,185	17,998	21,052	24,357	27,924	31,692				
Ordinance Reach Turning Basin					1,193,600	1,253,007	1,311,876	1,372,696		
Total	373,481	445,543	909,247	1,419,797	3,376,508	4,485,851	4,920,434	5,262,767	5,676,935	5,982,853

6.2.3 Proposed Dike Raise to Accommodate Current and New Work Volumes[21].

A 50 year dredged volume was calculated, as well as the new work volume for the upper harbor deepening to 56'. The total capacity shortfall at Clouter Creek DA is approximately 64 million cubic yards (mcy). With a total acreage of 1475 at Clouter Creek DA, a raise of 26.9' would be required to place all the material for the 50-year dredge volume. This excludes the extra capacity that is gained from utilizing the material from inside the DA to complete the dike raises. Numerous dike raises will be required to gain a 50-year capacity for Clouter Creek DA, with a final top elevation of approximately 50' (NAVD88). Each raise will be approximately 6'-7' in height.

[21] Data table from SAC, Operations Branch, circa September 2012. As per Caleb Brewer, maintenance dredging material must also be accounted for in disposal to Clouter Creek. He specifically mentions that "…Going back and adding in the areas that are not being studied for deepening, but material still goes to Clouter Creek is where the 837,216 cubic yards per year comes from. The original yearly average of 837,216 cy yr is the correct average for all reaches in which material is disposed of in Clouter Creek".

High strength geotextile would be placed with every dike raise to ensure the Factor of Safety (F.S.)[22] remains above 1.3. Each raise will be analyzed for slope stability and settlement prior to the designing of the raise. For the North, Highway, Middle, and South cells, each raise would also include a step-in, placing the next dike raise to the inside toe of the previous raise, as well as a fifty foot berm placed to the inside of the cell. The cross dike between the North and Highway cells, Highway and Middle cells, and Middle and South cells would be raised along the centerline.

6.3 Subsurface Investigation

Historical data was researched and data deficiencies were identified in order to locate areas on Clouter Creek DA which require further subsurface data. In October 2012, Cone Penetration Testing (CPT) was performed in those areas where data was deemed insufficient. Standard Penetration Testing (SPT) was performed in November and December 2013 at the previous CPT locations.

6.3.1 Field Methods

6.3.1.1 Cone Penetration Testing (CPT). In December 2012, the U.S. Army Corps of Engineers, Savannah District, performed cone penetration testing (CPT) on Clouter Creek Disposal Area. The CPT is also a widely accepted test method of *in situ* testing of foundation soils (ASTM D 5778) and provides a relatively inexpensive and rapid means for determining subsurface conditions. An instrumented conical shaped probe (60° cone tip, 10 centimeters in diameter, with the friction sleeve area 150 centimeters in diameter) is pushed into a soil deposit at a controlled rate of 2 cm/sec at each location to the termination depth. Depth of penetration is measured by an optical encoder, and is verified by manually measuring the depth of penetration and comparing the result to the final sounding depth measured by the encoder. The tip of the cone was instrumented to measure tip resistance (q_c) using strain gauges, while the attached sleeve was instrumented to measure friction (f_s) as the cone was advanced. The cone was also equipped with a pore pressure transducer to measure induced pore pressure or seismic sheer wave velocities (u_2) at discrete depth locations. Induced pore pressure is the excess pore water pressure generated by the probe displacing saturated soil. Low permeability soils will generate relatively high induced pore pressures, while high permeability soils will generate relatively low induced pore pressures. High permeability soils will generally show induced pore pressures that closely mirror hydrostatic pressures (u_0). The tip resistance, sleeve friction, and pore pressure were used to develop a profile of correlated soil type with depth. Output quantities for both sleeve friction and tip resistance are simultaneously recorded in units of tons per square foot per foot of depth. CPT testing provides a detailed record of cone resistance which is useful for evaluation of site stratigraphy. The use of the friction sleeve and pore-water pressure element is used to estimate soil classification and engineering properties of soils.

CPT testing was performed on 16 predetermined transects along the perimeter of all 4 cells of Clouter Creek Disposal Area (Figure B-71 and Figure B-72). Each transect consisted of 5 boring

[22] **Factor of safety** (**F.S.**) is a term describing the structural capacity of a system beyond the expected loads or actual loads. F.S. describes how much stronger the system is than it needs to be for an intended load. Safety factors are calculated using detailed analysis because comprehensive testing is impractical; however, the structure's ability to carry the load must be determined to a reasonable accuracy.

locations. These locations were: inside and outside embankment toe, inside and outside slope, and the crest. Of the 80 proposed CPT locations, only 67 were completed due to inaccessibility of the slope or toe locations. Several transects had steep outer slopes that dropped off to the marsh. In the instances where there was inadequate space to obtain all 5 testing locations, as many locations were tested as possible, allowing for the maximum collection of data.

Upon completion, all CPT borings were backfilled with bentonite grout. All CPT locations were recorded using a Trimble GeoXH GPS unit. Elevation data was acquired via LIDAR data provided by U.S. Army Corps of Engineer, Charleston District.

Figure B-60. Northern transect locations for Clouter Creek Disposal Area.

Figure B-61. Southern transect locations for Clouter Creek Disposal Area.

6.3.1.2 Standard Penetration Testing (SPT). In November and December, 2013, the U.S. Army Corps of Engineers, Savannah District, performed Standard Penetration Testing (SPT) on Clouter Creek DA. The test provides an indication of the relative density of granular soils, such as sand and gravel. Soil strength parameters derived from the test are generally considered approximate, but they are deemed acceptable given the widespread use of the method and it's relatively low cost. Correlation between the blow-count (N-value) and soil strength properties tends to be greater in sandy soils than in clayey soils. Despite this, the test method is used extensively to quantify soil properties for geotechnical engineering design.

SPT testing involves driving a standard thin-walled, 24-inch long, 2-inch OD/1-3/8-inch ID, splitspoon sampler a total depth of 18-inches into undisturbed soil. The driving energy for is imparted to the sampler (and length of drill rod) from the blows of a 140-lb hammer free-falling 30-inches. The number of blows to drive the sampler in three 6-inch increments is recorded. The first 6-inches of penetration is considered to be the seating drive. The sum of the number of blows required for the second and third 6-inches of penetration is termed the "standard penetration resistance" or the "N-value". The blows are applied and counted for each of the 6-inches until 18-inches of penetration is achieved. The test is terminated if: a total of 50- blows have been applied during any one of the three 6-inch increments, a total of 100-blows have been applied, or there is no observable advance in the sampler during the application of 10 successive blows of the hammer.

SPT testing was performed on eighteen predetermined locations along the perimeter of all 4 cells of Clouter Creek Disposal Area (Figure B-71, Figure B-72, and Figure B-73). Of the proposed thirty-two SPT locations, only eighteen were completed due to timeline and funding constraints. Thirteen of the SPT holes were located where CPT testing was previously performed in 2012. The remaining five SPT holes were located at new locations around Clouter Creek DA.

SPT testing was performed in accordance with ASTM D1586, as well as ER-1110-1-1807. Each SPT boring was advanced by using a mud rotary auger with cleanout to the top of the next sample. Each boring began at the ground surface and was advanced in drive increments of 1.5-feet to -73.5 ft NAVD88. The first SPT was taken at a depth of 2-feet and then on 5-foot centers to the bottom of the hole. After each sample was taken, the splitspoon sampler was washed to prevent cross contamination with the next sample. An inspector from SAW was on site during the drilling operations to visually classify the soils and record the SPT blow counts at each 18-inch drive. The splitspoon samples were sealed in jars and taken to the SAD laboratory at the end of the sampling effort. A total of 270 splitspoon samples were collected from the SPT endeavor.

SPT holes were backfilled with grout by inserting PVC tremie pipe to the terminal depth. The tremie pipe was then filled with bentonite grout weighing approximately 100 lbs/ft^3 and then retracted, keeping the pipe topped off with grout until all sections were brought to the surface. All SPT sampling locations and elevations were recorded using a Trimble VRS GPS unit.

6.3.1.3 Undisturbed Sampling. At selected SPT boring locations, an adjacent “sister” UD test boring was advanced within 10’ horizontally from the SPT boring location for the purpose of collecting undisturbed samples. The undisturbed samples were labeled SPT-13-CC-X UD-x where “x” represents the corresponding SPT numbering and undisturbed sample number. The depth interval, date, and time were also identified for each sample. The undisturbed sample depths were determined at discretion of the SAW inspector, based on CPT data, as well as field classification results of soils at certain SPT locations. Undisturbed sampling was performed in accordance with ASTM D1587. The thin-walled sampler tubes have an outside diameter of 3-inches and a total length of 30-inches. The undisturbed hole was advanced to the desired elevation using a mud rotary auger. The thin-walled samplers were then pushed for a penetration of 28-inches. After a thirty minute wait, the thin-walled sampler tube was removed from the boring, the recovery was measured, and the ends were sealed with wax and plastic caps. The tubes were labeled for orientation (top, bottom) and identification prior to being transported to the laboratory. Eighteen undisturbed samples were obtained, with some holes having two undisturbed samples taken and others having one undisturbed sample taken.

Figure B-62. SPT Boring locations for 2013 Clouter Creek Subsurface Investigation.

6.3.2 Laboratory Methods

6.3.2.1 ASTM D2216. Laboratory Determination of Water Content of Soil and Rock Mass. This test method covers the laboratory determination of the water (moisture) content by mass of soil where the reduction in mass by drying is due to the loss of water. For many materials, water content is one of the most significant index properties used in establishing a correlation between soil behavior and its index properties. The water content soil is used in expressing the phase relationships of air, water, and solids in a given volume of material. In fine-grained (cohesive) soils, the consistency of a given soil type depends on its water content. The water content of a soil, along with its liquid and plastic limit is used to express its relative consistency or liquidity index.

6.3.2.2 ASTM D2435. One-Dimensional Consolidation Properties of Soils Using Incremental Loading. This test method determines the magnitude and rate of consolidation of soil when restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. This test method is most commonly performed on undisturbed samples of fine grained soils naturally deposited in water. The data from the consolidation test are used to estimate the magnitude and rate of both differential and total settlement of earthen fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.

6.3.2.3 ASTM D2488. Description and Identification of Soils. This test method is used to identify soils based on visual examination and manual tests. Using visual examination and simple manual tests, soils can be identified using the classification group symbols and names. The descriptive information can be used to describe a soil to aid in the evaluation of its significant properties for engineering use.

6.3.2.4 ASTM D2850. Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils. This test method covers determination of the strength and stress-strain relationships of a cylindrical specimen of undisturbed cohesive soil. Specimens are subjected to a confining fluid pressure within a confined chamber. No drainage of the specimen is permitted during the test. The specimen is sheared in compression at a constant rate of axial deformation, without drainage. The compressive strength of a soil is determined in terms of the total stress; therefore, the material strength depends on the pressure developed in the pore fluid during loading. Fluid flow is not permitted from or into the soil specimen as the load is applied; therefore, the resulting pore pressure and strength differs from that developed in the case where drainage can occur.

6.3.2.5 ASTM D4318. Liquid Limits, Plastic Limits, and Plasticity Index of Soils. This test method is used to characterize the fine-grained fractions of soils. The liquid limit, plastic limit, and plasticity index of soils are also used with other soil properties to correlate with engineering behavior such as compressibility, hydraulic conductivity (permeability), compatibility, and sheer strength.

6.3.2.6 ASTM D4767. Consolidated Undrained Triaxial Compression Test for Cohesive Soils. This test method covers the determination of strength and stress-strain relationships of a cylindrical specimen of an undisturbed saturated cohesive soil. Specimens are isotropically consolidated and sheared in compression without drainage at a constant rate of axial deformation. The shear characteristics are measured under undrained conditions and are applicable to field conditions where soils that have been fully consolidated under one set of stresses are subjected to a change in stress without time for further consolidation to take place, and the field stress conditions are similar to those in the test method. The shear strength determined from the test is used in embankment stability analysis, earth pressure calculations, and foundation design.

6.3.2.7 ASTM D6913. Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis. This test method is used to determine the particle-size distribution (gradation) of a soil sample. A representative specimen is obtained from the sample after oven-drying. The specimen is sieved in its entirety, using a single sieve-set sieving. After the dry weight of the total sample was obtained, the sample was soaked in a dispersing agent. Once the samples had dispersed they were washed over a No. 200 sieve. The samples washed over the No. 200 sieve were then oven dried again and the dry weight after the No. 200 wash was recorded. If the sample weights indicated that over half of the material had passed the No. 200 sieve then no further testing was performed. However, if more than half of the sample was retained on the No. 200 sieve then the remaining portion of the sample was subjected to full sieve analysis after drying.

6.4 Settlement and Stability

6.4.1 Seepage Analysis

6.4.1.1 SEEP/W. Steady-state seepage analysis was performed using GeoStudio's SEEP/W, a two dimensional finite element modeling program. The phreatic surface and pore-pressure distribution was modeled for each dredging cycle after every raise for the fifty year life of the dike (Figure B-74). Levee cross sections were developed using subsurface data from the Cone Penetrometer Testing (CPT) data generated from the 2012 subsurface investigation and the 2013 as-built drawings supplied by the Charleston District from the 2012 LIDAR[23] topographic survey, then converted to finite element meshes. Hydraulic conductivity functions were defined, boundary conditions were applied, and seepage conditions were predicted for various dredging water elevations.

6.4.1.2 Seepage Analysis Assumptions and Input Parameters. For the preliminary designs, the dike profiles were modeled from the 2013 Clouter Creek Levee cross sections. These cross sections were developed from the 2012 LIDAR topographic survey conducted by the Charleston District (SAC). Four cross sections were modeled utilizing this data: the North cell at N389698.4, E2325489, the cross-dike between the North Cell and Highway Cell at N388326.8, E2323660, the Highway Cell at N386298.5, E2325713, and the Middle Cell at N382730.5, E2323906. After a site visit to Clouter Creek DA, it was discovered that the existing data did not match current conditions at the Middle Cell, and that analysis was terminated. The North Cell was only modeled for the first dike raise to elevation 26-feet. A 3H:1V inside slope and a 2H:1V slope was modeled for each raise, with high strength geotextile being placed at the ground level of each raise. The crest width is sixteen feet wide for each raise, and a fifty foot berm approximately three feet high is placed to the inside of the dike for stability. Each dike raise will be approximately six to seven feet. Dredged material taken from the inside of the disposal area

[23] Light detection and ranging (LIDAR) is a remote sensing technology that measures distance by illuminating a target with a laser and analyzing the reflected light. It is commonly used to make high resolution survey maps.

will be used to raise the dike. With each dredging cycle, two feet of freeboard will be modeled from the top of the dike.

SEEP/W inputs consist of cross sectional geometry, hydraulic conductivity and boundary conditions for the flow domain. Output results from SEEP/W consist of phreatic surface, head distribution, hydraulic gradient, flow directions and flow quantities within the flow domain. Each soil layer was assigned a vertical permeability (k_v) value based on experience with soil types and laboratory permeability tests. The horizontal coefficient of permeability (k_h) of each layer was assumed to be one to two times the vertical permeability. The seepage model follows steady-state conditions, with water surface elevations (headwater) at the crest of the dike system.

6.4.1.3 Seepage Analysis Results. As determined by SEEP/W, the seepage pore water pressure within the dike was minor. The phreatic surface exits near the landside toe of the slope with each dredging cycle (2-feet of freeboard). Lateral hydrostatic forces and seepage gradients within the dike and underlying foundation indicate the overall stability of the existing dike is acceptable.

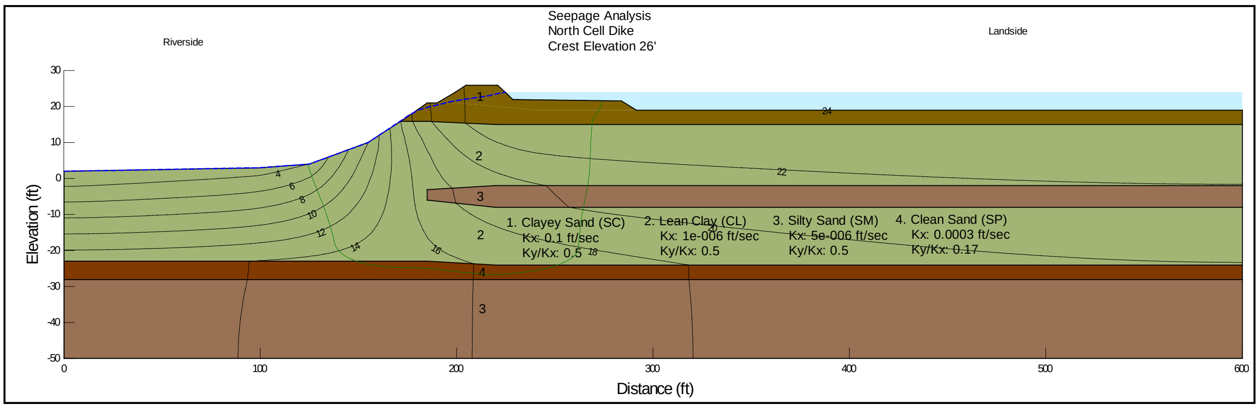


Figure B-63. Seepage analysis of Clouter Creek Disposal Area North Cell

6.4.2 Stability Analysis

6.4.2.1 SLOPE/W. Undrained slope stability analyses were performed using GeoStudio's SLOPE/W, a two dimensional finite element modeling program. SLOPE/W's formulation is based on the general limited equilibrium method, and uses an iteration scheme to find the critical slip surface and the corresponding minimum factor of safety. The factors of safety for sliding (block) and circular modes of failure were calculated in the analyses. The factor of safety for both circular and wedge failures were modeled for each dredging cycle after every raise for the fifty year life of the dike (Figure B-75 and Figure B-76). Dike cross sections were developed using subsurface data from the Cone Penetrometer Testing (CPT) data generated from the 2012 subsurface investigations and the 2013 as-builts supplied by the Charleston District from the 2012 LIDAR topographic survey data. Soil strength functions were defined, and slip surfaces were specified for each dike raise. Two types of slip surfaces were utilized for each dike raise, entry and exit (also referred to as circular failure) slip surfaces and block specified (also referred to as wedge failure) slip surfaces. The same cross section were used for both the SLOPE/W analysis and the SEEP/W analysis (North Cell, and the cross-dike between the North Cell and Highway Cell).

__6.4.2.2 Stability Analysis Results.__ As determined by SLOPE/W, the factor of safety (F.S.), decreases with each subsequent dike raise to the projected 50-year life cycle elevation of Clouter Creek DA. Utilizing geotextile into the design of the dike increases the factor of safety, however, the factor of safety falls below the minimum of 1.3 (EM 1110-2-1913, *"Design and Construction of Levees"*, dated 30 APR 2000) for end of construction after elevation 38'. Foundation preparation is recommended prior to raising the dike to ensure less future settlement and greater stability of the dike.

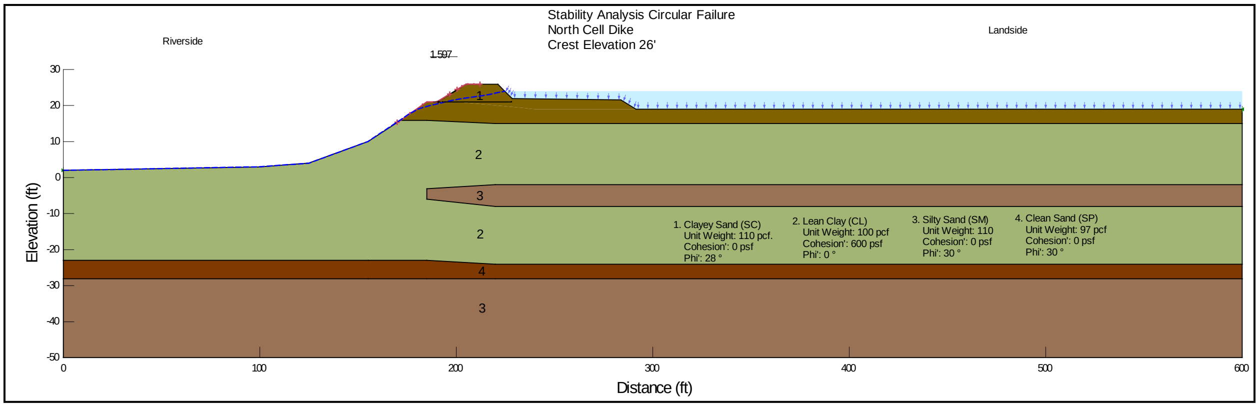


Figure B-64. Cirular failure, North Cell Dike, elevation 26'.

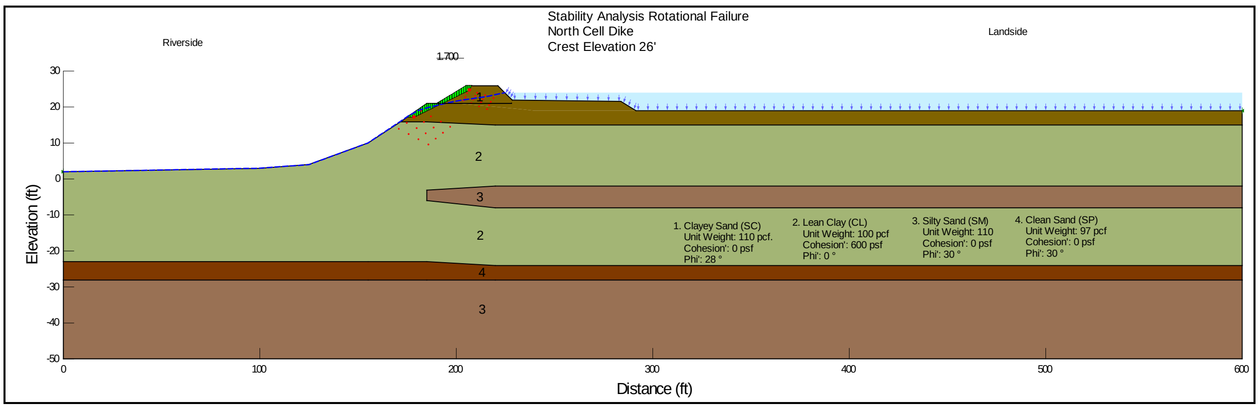


Figure B-65. Sliding failure, North Cell Dike, elevation 26'.

IX. REFERENCES CITED

Aucott, W.A., and Speiran, G.K.., 1985, Potentiometric surfaces of the Coastal Plain aquifers of South Carolina prior to development: U.S. Geological Survey Water-Resources Investigation Report 84-4208, 5 sheets.

Brainard, R., Smith, P., Drolet, L., 2009, Daniel Island Surprise-sand lens lurking in Cooper Marl, Charleston, S.C., in Rapid Excavation and Tunneling Conference Proceedings; Society for Mining, Metallurgy, and Exploration (SME), 20 p.

Colquhoun, D.J., 1974, Cyclic surficial stratigraphic unit of the Middle and Lower Coastal Plains, central Couth Carolina, in Oaks, R.Q., Jr., and DuBar, J.R., eds., Post-Miocene stratigraphy, central and southern Atlantic Coastal Plain : Logan, Utah, Utah State University Press, p. 179-190.

Corrado, J.C., J.C. Weems, R.E., Hare, P.E., and Bambach, R.K., 1986, Capabilities and limitations of applied aminostratigraphy, as illustrated by analysis of Mulinia lateralis from the Late Cenozoic marine beds near Charleston, South Carolina: South Carolina Geology, v. 30, no. 1, p.19-46.

Edwards, L.E., Gohn, G.S., Trail, J.M., Prowell, D.C., Bybell, L.M., Bardot, L.P., Firth, J.V., Huber, B.T., Frederiksen, N.O., MacLeod, K.G., 1999, Physical stratigraphy, paleontology, and magnetostratigraphy of the USGS-Santee Coastal Reserve Core (CHN-803). Charleston County, South Carolina: U.S. Geological Survey, Open-File Report 99-308, 66 p.

Gohn, G.S., Higgins, B.B., Smith, C.C., and Owens, J.P., 1977, Lithostratigraphy of the deep corehole (Clubhouse Crossroads Core Hole #1) near Charleston, South Carolina, in Rankin, D.W., ed., Studies related to the Charleston, South Carolina, earthquake of 1886-a preliminary report: U.S. Geological Survey Professional Paper 1028-E, p59-70.

Harris, S.M., Gayes, P.T., Kindinger, J.L., Flocks, J.G., Krantz, D.E., and Donovan, P., 2005, Quaternary geomorphology and modern coastal development in response to an inherent geologic framework: an example from Charleston, South Carolina: Journal of Coastal Research, v. 21-1, p. 49-64.

Harris, W.B., Zullo, V.A, and Baum, G.R., 1979, Tectonic effects on Cretaceous, Paleogene, and early Neogene sedimentation, North Carolina; *in* Baum, G.R., B.W. Harris, and V.A. Zullo, V.A., eds., Structural and stratigraphic framework for the Coastal Plain of North Carolina: Carolina Geological Society and Atlantic Coastal Plain Geological Association, Field Trip Guidebook, p. 17-29.

Hockensmith, B.L., 2001, Potentiometric map of the Floridian aquifer and Tertiary sand aquifer in South Carolina-1998: South Carolina Department of Natural Resources Water Resources Report 23, 1pl.

Horton, J.W. and V.A. Zullo, eds., 1991. The Geology of the Carolinas: Carolina Geological Society Fiftieth Anniversary Volume, University of Tennessee Press, p. 263-273.

Miller, J.A., 1990, Groundwater atlas of the United States, segment 6, Alabama, Florida, Georgia, and South Carolina; Hydrologic Investigations Atlas 730-G, U.S. Geological Survey, 28 p.

Park, D., 1985, The groundwater resources of Charleston, Berkeley, and Dorchester counties, South Carolina, State of South Carolina, Water Resources Commission Report Number 139, 79p.

Riggs, S.R., W.J. Cleary, and S.W. Snyder, 1995. Influence of inherited geologic framework on barrier shoreface morphology and dynamics: Marine Geology, v. 126, p. 213-234.

Sloan, Earle, 1908, Catalogue of mineral localities of South Carolina: South Carolina Geological Survey Bulletin, 4th series, no. 2, 505 p.

Sohl, N.F., and J.P. Owens, 1991, Cretaceous stratigraphy of the Carolina Coastal Plain, *in* Horton, J.W. and V.A. Zullo, eds., The Geology of the Carolinas: Carolina Geological Society Fiftieth Anniversary Volume, University of Tennessee Press, pp.191-220.

United States Army Corps of Engineers, Charleston District, 2009, Charleston Harbor Dredged Material Management Plan (DMMP), Preliminary Assessment.

Ward, L.W., Blackwelder, B.W., Gohn, G.S., and Poore, R.Z., 1979, Stratigraphic revision of Eocene, Oligocene, and lower Miocene formations of South Carolina; South Carolina Division of Geology Geologic Notes, v. 23, no. 1, p 2-32.

Weems, R.E., and Lemon, E.M., 1993, Geology of the Cainhoy, Charleston, Fort Moultrie, and North Charleston quadrangles, Charleston and Berkeley Counties, South Carolina, U.S. Geological Survey, Miscellaneous Investigations Series, Map I-1935, 2 sheets.

Weems, R.E., and Lewis, W.C., 2002, Structural and tectonic setting of the Charleston, South Carolina, region: Evidence from the Tertiary stratigraphic record: Geological Society of America Bulletin; January 2002, v. 114, no. 1, p. 24-42.

<u>Online Resources:</u>
ASHEPOO-COMBAHEE-EDISTO (ACE) BASIN SOUTH CAROLINA
NERRS Website, sponsored by National Oceanic and Atmospheric Administration
http://nerrs.noaa.gov/Doc/SiteProfile/ACEBasin/html/envicond/grdwater/gwtext.htm
Accessed February, 2012.
Groundwater well data accessed from the South Carolina Department of Natural Resources, Hydrology Section website: http://www.dnr.sc.gov/water/hydro/data.html
Accessed December, 2011.
City of Charleston Water Resource Information accessed from the Charleston Water Service website: http://www.charlestonwater.com/water_history_part1.htm

Entrance Channel 2011 Bathymetry and Historical Borings

Entrance Channel Sub-Sections

Borings 1986-1999

Sampling Method

- Unknown
- SPT
- Rock Core
- Vibracore

Borings are described in terms of Bottom of Hole (BOH) depth and Unified Soil Classification System/or lithology

2011 Condition Survey

Bathymetry (MLLW)

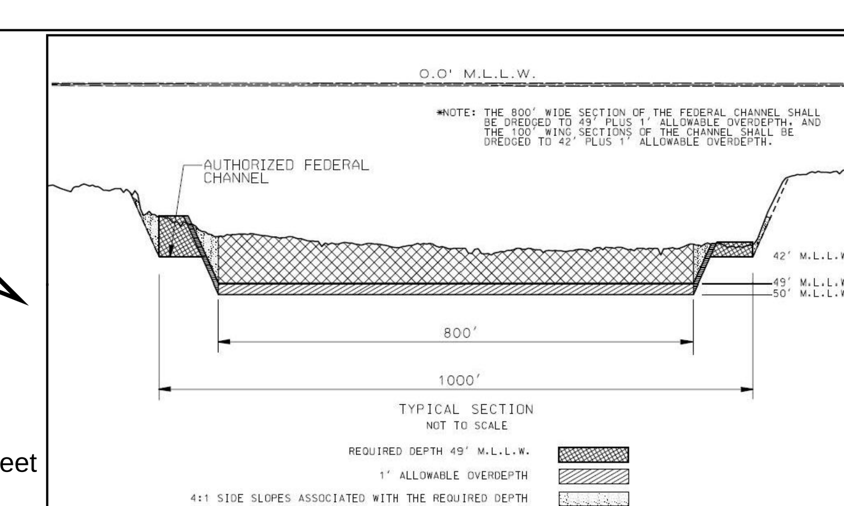
- 25.2 - 30
- 30 - 34
- 34 - 38
- 38 - 40
- 40 - 44
- 44 - 46
- 46 - 48
- 48 - 52
- 52 - 54
- 54 - 56
- 56 - 58
- 58 - 60
- 60 - 65
- 65 - 70
- 70 - 75

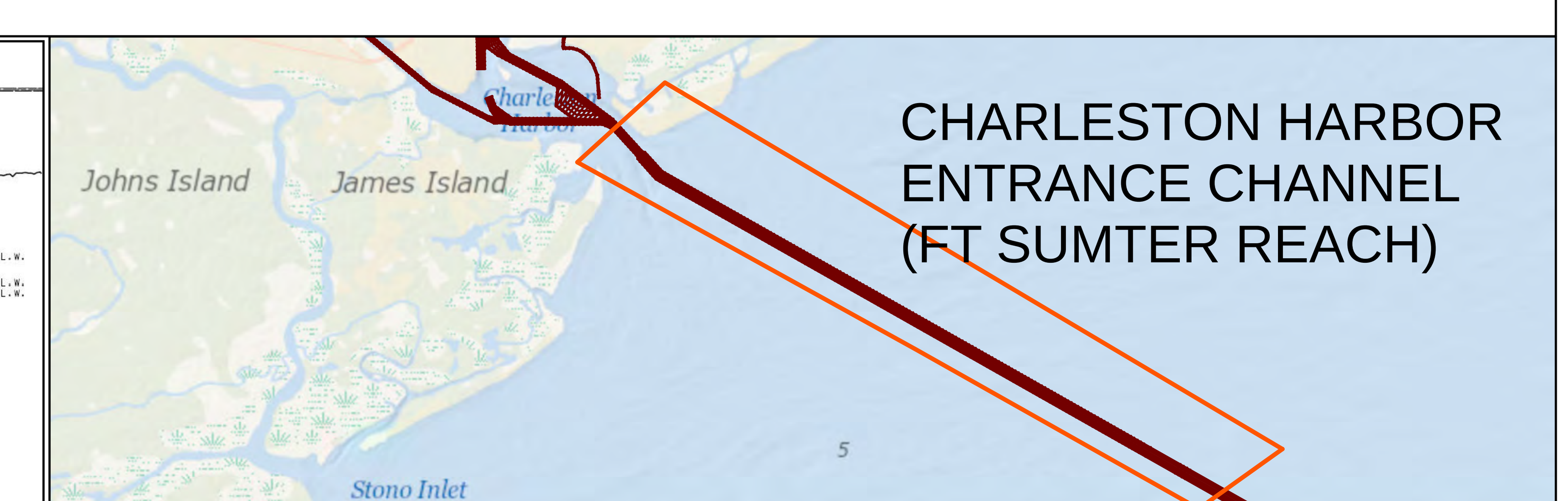
PLATE 1: CHARLESTON HARBOR ENTRANCE CHANNEL BATHYMETRY & HISTORICAL BORINGS 1986-1999 SURVEYED BY CHARLESTON DISTRICT

N

0 0.125 0.25 0.5 0.75 1 Miles

1,300 650 0 1,300 2,600 3,900 5,200 Feet





CHARLESTON HARBOR ENTRANCE CHANNEL (FT SUMTER REACH)